California High-Speed Train Project

Request for Proposal for Design-Build Services

RFP No.: HSR 11-16

Book 4, Part B, Section 6
Construction Package 1
Bridges and Elevated Structures Report

Prepared for
CALIFORNIA
High-Speed Rail Authority

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CALIFORNIA HIGH-SPEED TRAIN

Engineering Report

RECORD SET 30%
DESIGN SUBMISSION

Fresno to Bakersfield

Sierra Subdivision
Procurement Package 1
Structures Report

July 2012

CALIFORNIA
High-Speed Rail Authority

California High-Speed Train Project

Structures Report

Prepared by:

URS/HMM/Arup Joint Venture

July 2012

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1.0 Introduction

This report identifies the key features of each of the structures in the southern portion of Packages 1A, 1B, and 1C of the California High-Speed Train Project (CHSTP). The track structures are shown in Figure 1.0-1 below.

The environmental review of the Merced to Fresno and Fresno to Bakersfield sections of the CHSTP overlap in the city of Fresno. The environmental review, including mitigation measures adopted by the Authority, for Package 1A from station 10806+00 to 10970+00 and Package 1B (Figure 1.0-1) are contained in the Merced to Fresno Final Environmental Impact Report/Environmental Impact Statement (EIR/EIS). The environmental review for Package 1C is contained in the environmental documents for the Fresno to Bakersfield Section.

This report covers only the HST structures considered nonstandard or complex. CHSR Design Criteria provide the definition of *nonstandard* and *complex* structures.

The design criteria divide structures into a classification hierarchy as follows:

- Primary structures (structures that directly support the HST tracks)
- Secondary structures (all other structures)

Primary structures are subdivided by importance into the following:

- Important structures (structures designated by the Authority to be important)
- Ordinary structures (all other structures)

Primary structures are also classified by technical complexity as follows:

- Complex structures: Structures that have complex response during seismic events through
    - Irregular geometry
    - Unusual framing
    - Long spans
    - Unusual geologic conditions
    - Close proximity to hazardous faults
    - Regions of severe ground motion
- Nonstandard structures: Structures that do not meet the requirements for either standard or complex structures

Table 1.0-1 lists the structures in the southern portion of Package 1 of the Fresno to Bakersfield section of the HST, and indicates their classification under the above system.

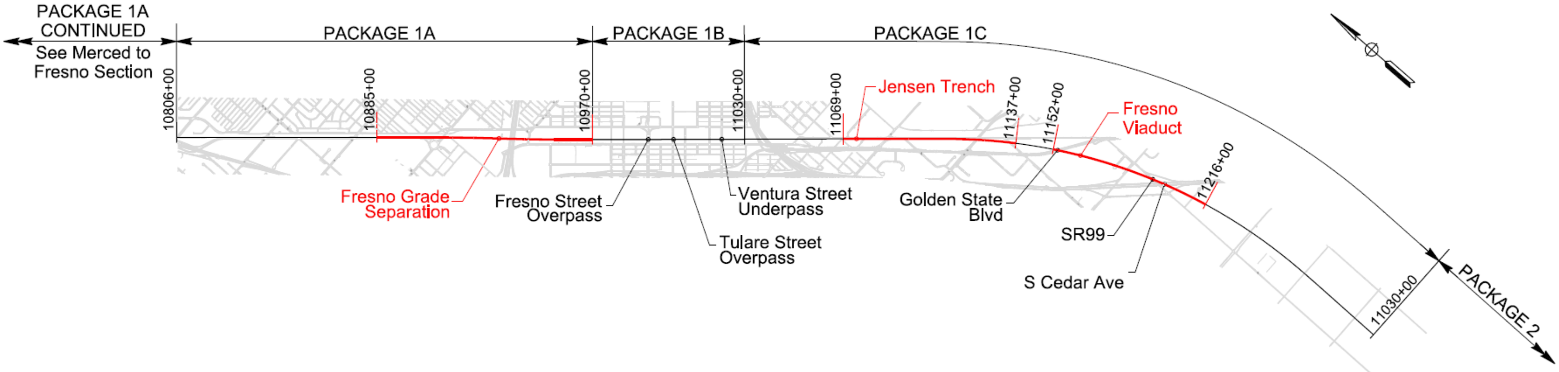


Figure 1.0-1
Key map of Package 1

Table 1.0-1
Structures and Components

Package Reference	Primary Structure Name	Structural Component	Location or Start Station on Alignment S	End Station on Alignment S
1A	Fresno Grade Separation	Reinforced concrete (RC) U-trough Primary - Nonstandard	10885+00	10909+30
1A	Fresno Grade Separation	Braced RC U-trough Primary - Nonstandard	10909+30	10920+20
1A	Fresno Grade Separation	Covered trench at SJVR Northern Spur Crossing Primary - Nonstandard	10920+20	10940+05
1A	Fresno Grade Separation	Braced RC U-trough Primary - Nonstandard	10940+05	10933+80
1A	Fresno Grade Separation	Covered trench at Dry Creek Canal Crossing Primary - Nonstandard	10933+80	10934+20
1A	Fresno Grade Separation	Covered trench at SJVR Southern Spur Crossing Primary - Nonstandard	10934+20	10935+20
1A	Fresno Grade Separation	Braced RC U-trough Primary - Nonstandard	10935+20	10935+95
1A	Fresno Grade Separation	Jacked box beneath SR 180 Primary - Nonstandard	10935+95	10939+40
1A	Fresno Grade Separation	Braced RC U-trough Primary - Nonstandard	10939+40	10941+90
1A	Fresno Grade Separation	RC U-trough Primary - Nonstandard	10941+90	10970+00
1B	Fresno Street Overpass	Two Span In-situ Box Underpass –Not in Contract	10991+70	10992+50
1B	Tulare Street Overpass	Two Span In-situ Box Underpass Primary - Standard	11001+53	11002+05
1B	Ventura Street Overpass	Two Span In-situ Box Underpass Primary - Standard	11020+50	11021+40
1B	Ventura Street UPR Bridge	Two span steel beam bridge Secondary – Non standard	11020+50	11021+40

1C	Jensen Trench	RC U-trough Primary – Nonstandard	11069+40	11140+00
1C	Fresno Viaduct Approach Ramp	MSE retained Embankment Primary - Standard	11141+00	11152+20
1C	Fresno Viaduct Golden State Boulevard	Steel Truss Primary - Complex	11152+20	11155+36
1C	Fresno Viaduct	Post tensioned spans Primary - Standard	11155+36	11191+47
1C	Fresno Viaduct South Cedar Avenue	Steel Truss Primary - Complex	11191+47	11195+02
1C	Fresno Viaduct SR 99 Undercrossing	Steel Truss Primary – Nonstandard	11195+02	11199+97
1C	Fresno Viaduct	Post tensioned spans Primary - Standard	11199+97	11216+02
1C	Fresno Viaduct Approach Ramps	MSE retained Embankment Primary - Standard	11216+02	11230+00
1C	Facility Access Structure	RC Box Underpass Primary - Standard	11218+00	11218+30
1C	Central Canal	RC Box Culvert Primary – Standard	11237+00	11237+40
1C	Viau Canal	RC Box Culvert Primary – Standard	11279+90	11280+10

1.1 Overall Design Assumptions for Preliminary Design

In carrying out the analysis, the designers have concentrated on the key aspects of the design stated in the analysis scope. These aspects are determined in many cases by satisfying the requirements of the relevant design criteria.

For the U-troughs, the requirements are concerned with the following:

- Structural adequacy
- Constructability and consideration of adjacent constraints
- Technical feasibility
- Design economy

For the bridge structures, the requirements include the following:

- Structural adequacy
- Seismic performance as specified in the seismic design criteria
- Interaction between track and structure to ensure that adequate provision is made for relative and absolute displacements between track and structure
- Constructability and assumed construction method
- Design economy

1.1.1 Structural Adequacy

For the U-trough, the designers performed preliminary calculations on a number of cross sections to demonstrate that the assumptions about section wall thickness, shoring wall thickness, and excavation sequence were reasonable.

The designers performed similar calculations for key components of the U-trough — specifically the jacked box and Dry Creek Canal culvert. The Dry Creek culvert itself is not an HST structure, but preliminary design was necessary to demonstrate that there was clearance for the U-trough to pass under the creek and for the San Joaquin Valley Railroad (SJVR) tracks to pass over the structure without compromising its hydraulic performance.

1.1.2 Seismic Performance

The seismic design criteria give the requirements for assessment of the seismic performance of structures. In terms of acceptability of the design, the requirements relating to seismic performance are Operability Performance Level (OPL) under the action of the Operating Basis Earthquake (OBE) and No-Collapse Level (NCL) of performance under the action of the Maximum Considered Earthquake (MCE):

NCL at MCE:

- No collapse
- Significant yielding of reinforcing steel
- Extensive cracking and spalling of concrete but minimal loss of vertical load carrying capacity in columns
- Large permanent deflections

OPL at OBE:

- Minimal impacts to HST operations
- No spalling of concrete
- Minimal permanent deformations

The seismic design criteria define the two design-level earthquakes as follows:

Maximum Considered Earthquake (MCE) - Ground motions corresponding to greater of:
(1) a probabilistic spectrum based upon a 10% probability of exceedance in 100 years (i.e., a return period of 950 years with 5% damping) and
(2) a deterministic spectrum based upon the largest median response resulting from the maximum rupture (corresponding to Mw) of any fault in the vicinity of the structure

Operating Basis Earthquake (OBE) - Ground motions corresponding to a probabilistic spectrum based upon an 86% probability of exceedance in 100 years (i.e., a return period of 50 years with 5% damping)

Response spectra for design have been the subject of separate studies (see also Appendix A Geotechnical Design Memorandum). The Engineering Management Team (EMT) has provided spectra from these studies for use in the preliminary design. These spectra are reproduced in Figure 1.1-1.

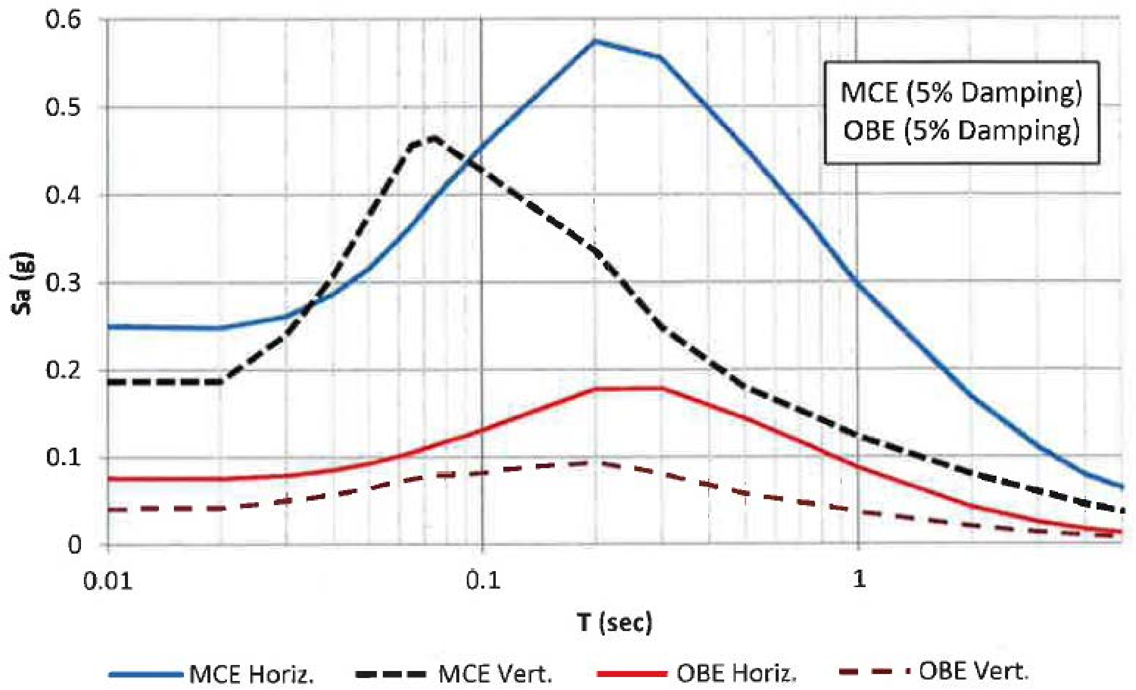


Figure 1.1-1
Design Response Spectra (Zone 4)

The seismicity in the Fresno area is categorized as Zone 4, which is the lowest category encountered on the Fresno–Bakersfield Section of the HST.

The peak ground acceleration (PGA) has been taken as the acceleration that corresponds to a period of 0.01 seconds — that is 0.0761g at OBE (red curve) and 0.2498g at MCE (blue curve). As these accelerations are less than 0.35g, in accordance with TM 2.9.10 clause 6.10.13, additional earthquake pressures can be disregarded for the design of buried structures such as the U-trough and the jacked box, provided that design for at-rest pressures is undertaken. However the structure should not collapse.

1.1.3 Dynamic Performance

Fundamental frequency checks have been carried out for the HST bridge structures in compliance with the requirements of the seismic design criteria. Details of this analysis are reported in the Seismic Design Plan, attached as Appendix B, and selected summary results have been provided in section 4.3.

1.1.4 Track Structure Interaction

The HST bridges within the southern portion of Package 1A considered in this report (Tulare Street and Ventura Street) are relatively short, approximately 98 feet. No critical interactions are expected along the length of these standard structures.

1.1.5 Assumptions Made for Preliminary Stage Design

The recommendations of the Geotechnical Design Memorandum (Appendix A) have been followed, including the following:

- Soil parameters ($\gamma_b, \emptyset, c_u$)
- Assumed groundwater levels
- The requirements of TM 2.3.2 (see also Section 2.2.1)

For the jacked box assessment and preliminary design, assumptions have been made concerning the following:

- Soil parameters for the State Route (SR) 180 embankment
- The percentage of ground loss (overbreak) that can be expected during excavation
- Groundwater levels

For the U-trough shoring wall, assumptions have been made concerning the following:

- The type of shoring that will be used
- Soil parameters
- Temporary construction surcharges
- Temporary brace positions, spacing, and stiffness
- The Kinder-Morgan hydrocarbon line has not been considered in the analysis undertaken; if not diverted, the permissible movements that it can tolerate (unknown at present) may influence the design and type of shoring that can be used in the vicinity

More detail concerning these assumptions is provided in the individual sections for each structure.

The DB contractor or jacking specialist should verify these assumptions based on the results of the ground investigation and any other investigation they may undertake.

1.1.6 Further Information Required to Develop the Design

It is expected that the DB contractor will wish to have more detailed information regarding key design issues. These issues include, but are not limited to:

- Borehole details along the length of the U-trough
- Results of soils testing (currently planned)
- Results of long-term monitoring of groundwater levels
- More detailed assessment of surcharge loading
- Detailed knowledge of access routes and timing of access to site
- Details of the location of overhead contact system (OCS) posts and wall mountings
- Detailed discussions with Fresno Irrigation District relating to timing and construction sequencing of the 96-inch storm drain diversion
- Approved schedule of road closures and durations for cross streets
- Detailed discussions with Caltrans about acceptable settlements of the SR 180 bridge in response to more detailed proposals regarding box jacking process and methodology
- Greater detail about utility crossings in order to plan the protection measures required
- Definite information from Union Pacific Railroad (UPRR) about acceptability of the Tulare Street undercrossing bridge and the methodology for installation of the new deck

The design has not allowed for temporary or permanent surcharges applied to land outside the right-of-way other than the known UPRR loadings described above. It is considered advisable that negotiations for the right-of-way should include conditions either for the permitted use of land adjacent to the U-trough that limit the loading that can be applied or that additional land is purchased so that its use can be controlled.

2.0 Fresno Grade Separation

The Fresno Grade Separation is a reinforced concrete (RC) U-trough structure that varies in depth from approximately 0 to 50 feet. The trench will be constructed at a part of the route where the right-of-way width is constrained by adjacent properties; this restricts the methods by which the structure can be built, effectively excluding open cut excavation.

The grade separation structure is composed of a number of subtypes:

- Reinforced concrete U-trough
- Reinforced concrete U-trough with permanent high-level bracing
- Cut-and-cover structure
- A section of tunnel that is to be constructed off-line and jacked into position through an embankment
- Utility crossing structures

2.1 Structure Importance Classification

TM 2.3.2 paragraph 2.2.1 defines all structures supporting the high-speed tracks to be primary structures because they will be required to be reinstated to allow resumption of train service after an earthquake.

This classification implies the following:

- Design life is 100 years
- Seismic design must comply with TM 2.10.4; however, the seismic design criteria for the Fresno Area indicate a PGA of less than 0.35g. In accordance with TM 2.9.10 clause 6.10.13, this means that additional earthquake pressures can be disregarded for the design of this structure
- When applying the AASHTO LRFD code, values for the importance, ductility, and redundancy factors, hI, hD, and hR have been chosen as follows:
  - Importance factor hI = 1.05
  - Ductility factor hD = 1.05 for strength calculations
  - Redundancy factor h_R = 1.05 for nonredundant elements, 1.0 otherwise

2.2 Key Design Features and Site Constraints

The grade separation is a simple RC U-trough (where the depth is such that additional permanent bracing is not required). These sections will be designed as rigid walls in accordance with TM 2.3.2 clause 6.4.3 which means that an “at-rest” earth pressure coefficient will be used instead of an “active” pressure coefficient. Appropriate load factors from the AASHTO LRFD code will be applied to give the design forces. The typical cross section of this configuration is shown in Figure 2.2-1.

The typical section indicates a 10-foot-high wall to the left of the section (east of the route). This collision protection wall provides protection for the HST route from intrusion by derailed trains from the adjacent UPRR. This wall has been added to the structure of the U-trough because the constrained width of the right-of-way in Roeding Park and other areas precludes providing this protection on an independent foundation. This wall is not required where the separation between the UPRR right-of-way and the HST right-of-way exceeds 102 feet.

To the right (west) side, the wall has been raised 3 feet above ground level to provide a nominal delineation of the edge of the trench. Additional fencing is required for fall prevention in most areas; this is not shown on the section. At the right-side boundary, access restriction fencing is provided on independent foundations to delineate the right-of-way.

Either concrete channels or swales provide drainage of the ground adjacent to the U-trough.

All surfaces of underground structures exposed to fill or water will be waterproofed.

The section also indicates OCS equipment, which is attached either to the top of the walls or to the side faces of the wall. As the U-trough deepens, it becomes more convenient to mount the OCS on the side faces of the walls. In areas where the height of the conductor or feeder cables is within 10 feet of the ground, there is potential for a touching hazard. In these areas, it is considered prudent to raise the height of the wall to 10 feet above ground level so that the OCS can be mounted on the wall face instead of the top.

The OCS equipment is not part of the civil engineering contract; however, knowledge of its location is required in order to finalize the design of the wall in these areas.

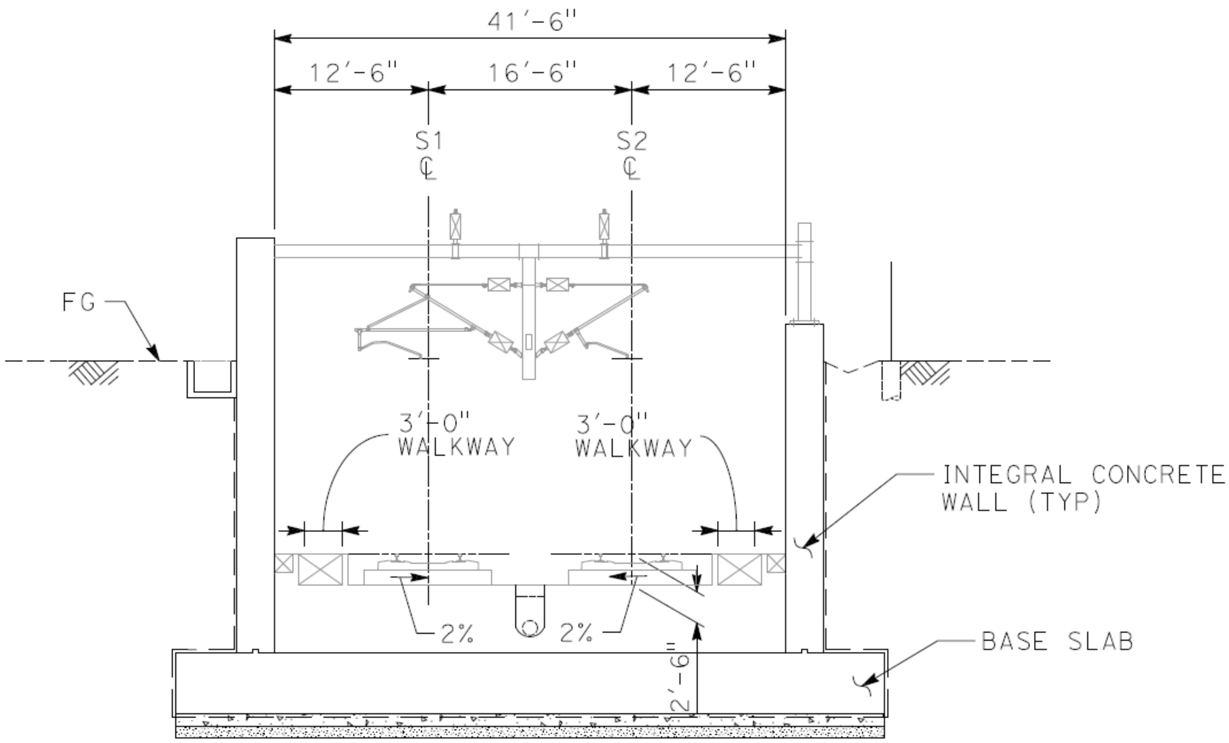


Figure 2.2-1
Typical Section of Unbraced U-Trough

Soil parameters used in the design have been based on historical geotechnical data along the HST Fresno subsection from State Routes 41, 43, and 99 as supplemented by City of Fresno residential development project records. Subsequent ground investigations have validated these assumptions and have provided additional data for Package 1C.

Where the depth of the trench exceeds approximately 30 feet from ground level to the top of rail, an unbraced section becomes difficult to achieve without excessively heavy reinforcement. Permanent bracing then becomes a more effective solution.

The minimum clearance requirements for the OCS system allow braces to be placed no lower than 27 feet above top of rail, which places the braces close to ground level at the start of the braced sections. As the trough continues to deepen, the braces maintain their clearance to the OCS. This also reduces the bending moments at the root of the wall.

At Dry Creek Canal and select utility crossings, a reduced clearance of 24 feet has been provided. This is subject to approval of a design variance.

Due to the additional stiffness provided by the brace, these sections must also be designed as rigid walls in accordance with TM 2.3.2 clause 6.4.3, using the “at-rest earth” pressure coefficient with appropriate load factors from the AASHTO LRFD code.

The typical section of this arrangement is shown in Figure 2.2-2.

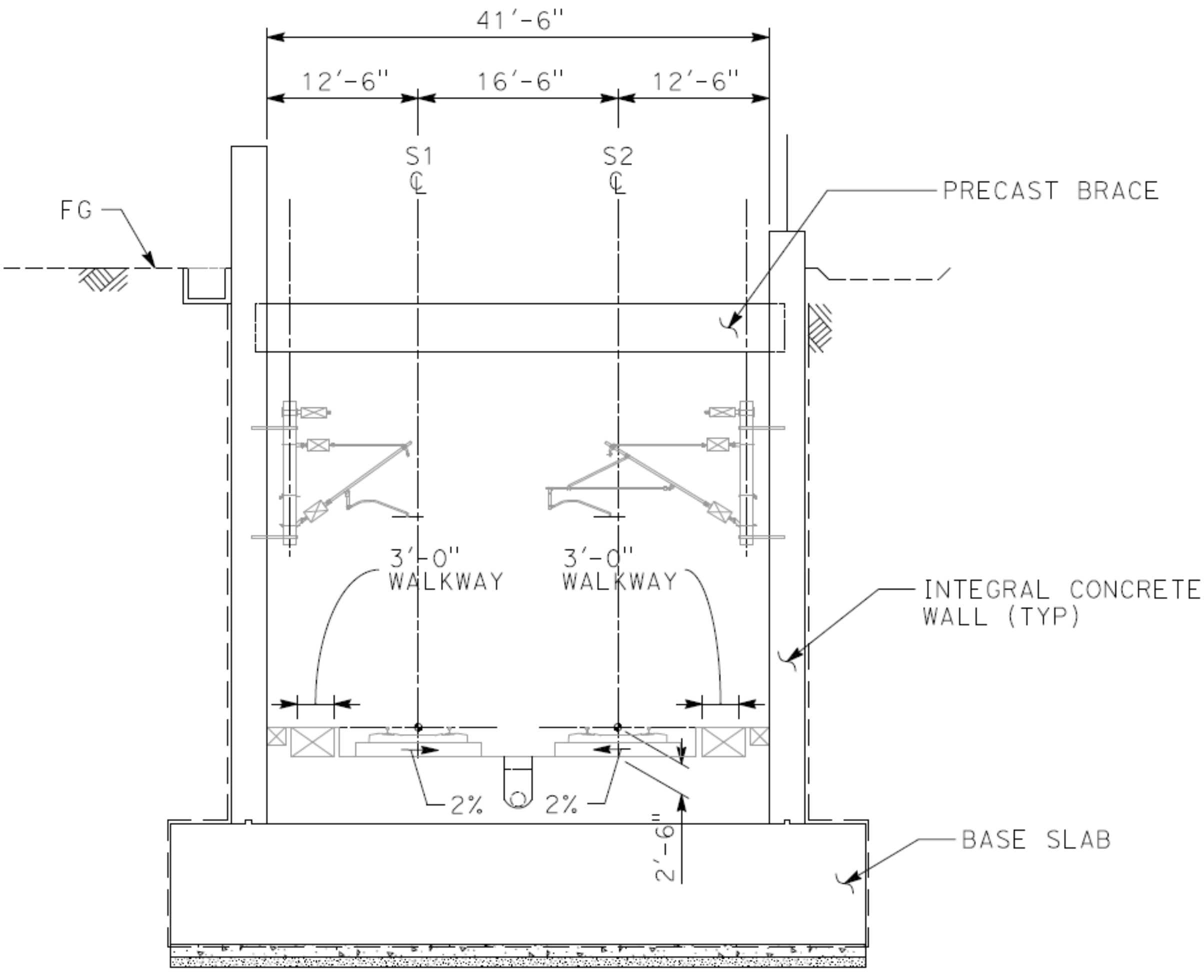


Figure 2.2-2
Typical Section of Braced U-Trough

2.2.1 Design Assumptions

2.2.1.1 Locked in Force from Shoring

In accordance with TM 2.3.4, the U-trough walls have been designed as rigid walls subject to at-rest earth pressures. In addition, where the walls will be restrained by permanent bracing, to allow for restraint forces that will be “locked-in” from the temporary bracing, the earth pressure calculated at the base of the wall has been assumed to act for the full height of the wall. This is similar to the pressures found in the design of temporary bracing to the excavation. The forces resulting from this assumption add approximately 25% to the forces that otherwise would be calculated.

2.2.1.2 Groundwater Level

Groundwater levels have been assumed to be generally below the level of the excavation except in areas where there is a ready water supply. These are assumed to be at detention basin RR2 and at Dry Creek. In these places, the water level is assumed to be 40 feet below ground level as recommended by the Geotechnical Design Memorandum (Appendix A). Adjacent to these areas, it is assumed that water level gradually reduces.

2.2.1.3 Surcharge Pressure

For the majority of the length of the U-trough, the right-of-way has a width of only 60 feet. To the east side, the right-of-way of UPRR abuts the HST right-of-way, and for approximately 1,000 feet, Roeding Park abuts to the west. Consequently, in these areas it is not likely that the construction surcharges specified in TM 2.3.2 clause 6.4.4 will be possible. Nor is it likely that future developments will add to the surcharge. In areas where the route passes between G Street and H Street, surcharges are possible because the right-of-way width is greater and because a number of properties taken by the route may include saleable parcels of land.

The UPRR tracks are generally within 30 to 100 feet of the U-trough for much of its length, and the possibility of additional surcharge from derailments exists.

At its current location, the UPRR adds little to the force applied to the wall. The maximum contact pressure of the Cooper E80 loading (driving wheels) is 1,882psf at the underside of ties. This pressure was applied to the wall using the Boussinesq formula. The resulting moment effect at the base of the stem was back-calculated to an equivalent uniform surcharge. This procedure has demonstrated that a uniform surcharge of 420psf (3.86 feet of fill) would be adequate allowance for the Cooper E80 load and any short-term derailment surcharge, unless the offset to the nearest track centerline is less than 20 feet.

Where the SJVR spur tracks cross the trough, a surcharge of 1,882psf has been applied.

Similarly, where adjacent land is available for potential development, a surcharge of 600psf as required by TM 2.3.2 clause 6.4.4 has been applied.

2.2.1.4 Collision Intrusion Barriers

A level of protection from a derailed UPRR train is provided by increasing the height of the U-trough wall to 10 feet. Collision forces have been considered in the design of the upper sections of the wall where forces are concentrated. The design has allowed for two forces as specified by the UIC leaflet 777-2R. In practice, the upper force of 112.4 kips applied at a 9.84-foot height is only critical in the upper sections of the collision wall itself. The lower force of 449.6 kips at a 3.24-foot height is generally critical for the upper parts of the trough wall.

In order to reduce the risk of significant impact events affecting the body of the U-Trough wall it is recommended that the wall section immediately below the collision intrusion barrier should be designed to a higher capacity so that impact effects are localized to the area above ground level.

2.2.1.5 Methods of Counteracting Buoyancy

The concept for the trough is a development of the 15% stage design concept. The concept assumes that the temporary excavation for the trough is retained by shoring walls that are either removed or abandoned after the trough is constructed. For U-trough structures like this, rising groundwater levels are a threat because of the risk that the structural will float. This has happened in some rare cases.

A number of counterstrategies were considered in the development of the design, including the following:

- **Heels**
  The directive drawings indicate a heel detail, which means that in order to float, the buoyancy forces must overcome the weight of backfill over the heel in addition to the weight of the trough

itself. This detail is designed for situations where the structure is constructed in open cut or at least with greater available space. It has not been considered suitable for this trough.

- **Thick bases**
  A second way to counteract buoyancy is to make the structure heavier. This is commonly achieved by thickening of base slabs. In the case of this structure, however, it would be necessary to have base slabs over 20 feet thick in some locations. This would be excessively costly, both in extra concrete and in extra excavation.
- **Attachments to walls**
  When a U-trough structure has a permanent shoring wall, it is common for the U-trough structure to be connected to the shoring walls using dowels or reinforcing bars drilled and post-fixed to shoring wall. The shoring wall then resists the uplift forces from buoyancy through skin friction with the ground. In the case of this trough, this option was discounted on the basis that the directive drawings require the trough to be "fully-tanked," i.e., to have continuous waterproofing membrane around its external surface. Dowel bars or reinforcement would have punctured this membrane, compromising the seal
- **Permanent walls**
  A development of the previous option is to combine the functions of the shoring walls with that of the permanent wall. This would limit the type of wall to either secant or diaphragm walling because of the need to maintain watertightness. The base slab of the trough would be constructed as the proposed U-trough but would be doweled to the diaphragm or secant pile wall at the edges. This option has not been pursued for the same reasons as above. However, a DB contractor may wish to develop this option further.
- **Micropiles**
  This option considers the construction of Micropiles of approximately 1-foot diameter at intervals along the length of the trough. Calculations suggest that one pile 35 feet long under each track at intervals of 5 feet would be sufficient to counteract the expected buoyancy forces. This method uses approximately 1/70th of the volume of concrete that would be required by thickening the base slabs.
- **Change watertightness requirement**
  There is a clear requirement that the trough should be watertight. This is expressed in the directive drawings that require waterproof membrane. However, if this requirement were to be relaxed to permit some water inflow, it could have the following effects on the design:

Benefits

- Open up the range of wall types to include contiguous bored piles
- Remove all risk of buoyancy
- Reduce the need to design for water pressures
- Remove the need for waterproofing membrane

Drawbacks

- Need to increase the size of drainage pipes and sump storage capacity and pumping
- Increased pump running cost

Risks

- At some future date if groundwater rises to a level that the inflow cannot be carried by the drainage and sumps, it may be necessary to install a cut-off grout curtain to reduce inflow or to install a pumped groundwater abstraction system, if permitted
- Retrofitting the above works would be expensive and disruptive to operations

Of the options considered, Micropiles are thought to be the most economical and effective option for restraining the U-trough.

Information about actual groundwater levels has since been obtained, and this suggests that buoyancy resistance is not required.

2.2.2 Key Constraints

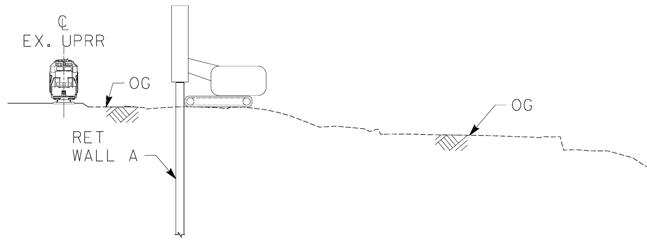
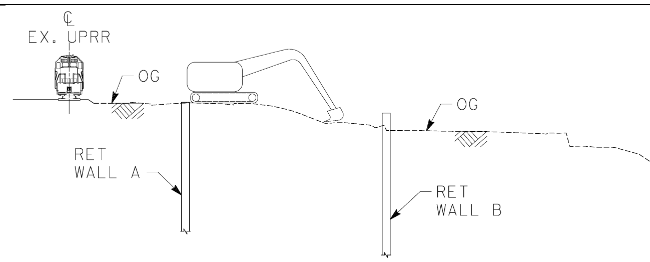
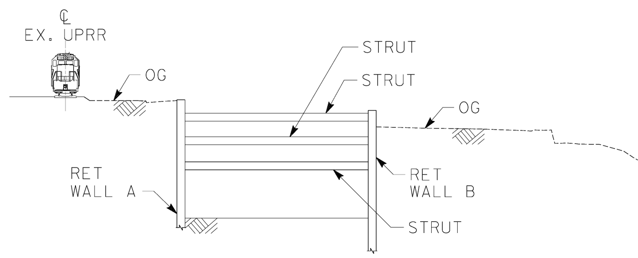
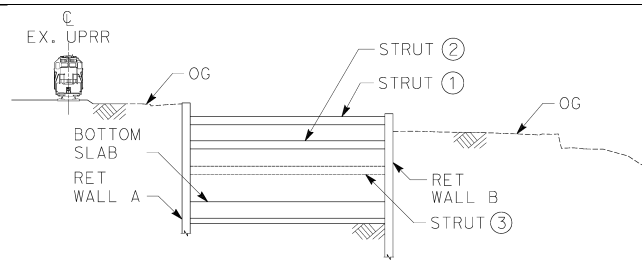
Some constraints apply to the trench as a whole, while others are design and construction constraints that may apply to only one component of the structure. The key constraints on the trench are:

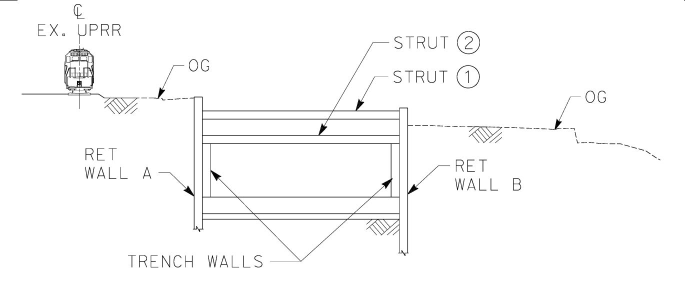
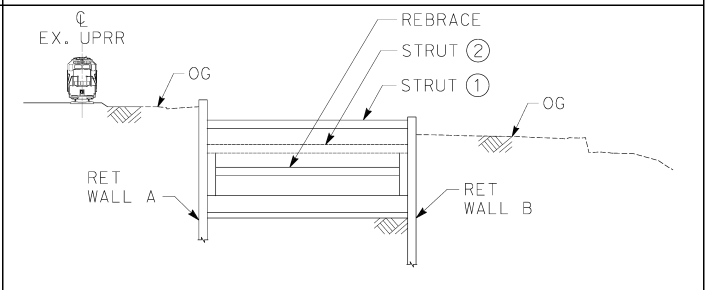
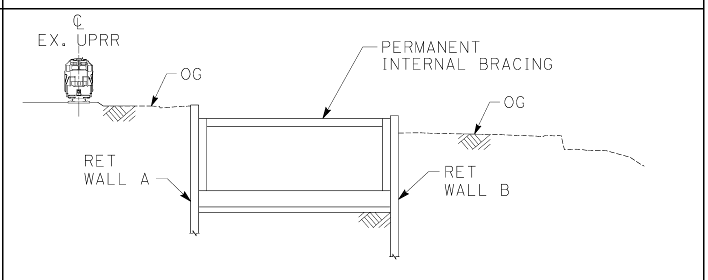
- The width of the right-of-way is generally less than 100 feet. Adjacent to Roeding Park, it is approximately 60 feet, and in the area of SR 180, 80 feet. As the required minimum width for the track alignment and equipment is 42 feet, this at worst leaves only 18 feet for the following:
    - All permanent retaining walls
    - Temporary shoring required for construction
    - Boundary controls required to delineate the right-of-way boundary (boundary fence, intrusion protection, intrusion detection, etc.)
    - Drainage (swales and channels)
    - The 96-inch storm drain diversion
    - Drainage sump access
    - Emergency egress stairs

This width limitation is particularly critical in the Roeding Park area. Consequently, the method and sequence of construction of all parts of the trench should be developed in a carefully planned sequence to avoid the risk that parts of the site may become inaccessible for the completion of subsequent operations.

The assumed construction stages are shown in the Table 2.2-1. Work is subject to restrictions from both an operating railroad and Roeding Park.

Table 2.2-1
Assumed Construction Stages

Construction Stage	Stage Diagram
Stage 0 Install shoring walls from within HST right-of-way.	
Stage 1 Excavate to below first brace level.	
Stage 2, 3, etc. Excavate under previous stage bracing (using low-height excavators if required). Install bracing as needed.	
At required depth Place mudmat and waterproofing membrane. Fix base reinforcement. Cast U-trough base slab.	

Stage 4 Remove lower brace (3). Fix waterproofing membrane. Fix wall reinforcement. Cast wall to part height.	
Stage 5 If required, reinstall lower brace. Repeat Stage 4 until full height achieved.	
Stage 6 Install permanent brace in deeper sections of U-trough. Cast collision wall if required.	

- The preliminary design has developed this concept, and in order to confirm the feasibility of the proposed structure, calculations of the shoring wall requirements were carried out following the proposed construction sequence through to the permanent case. This work has confirmed that, in principle, a 3-foot wall thickness is feasible for the U-Trough, although in some locations it may need to be heavily reinforced or require local thickening of the wall.
- The proximity of the UPRR tracks means that most of the length of the trench on the side adjacent to UPRR right-of-way requires collision protection, as defined by the TM 2.7.5 draft, dated July 18, 2011.
  This barrier has been added to the top of the trench wall where possible because (1) the trench provides a foundation that is sufficiently robust to carry the accidental forces and (2) to provide a completely separate foundation in this area would be difficult because of limited space and difficult access post-construction.
- UPRR will have requirements to protect the track during construction of the trench. At a later stage of the design, the contractor should address these requirements, which could include speed restrictions to trains, additional derailment protection, limits on the use of tall plant, and limits on the hours of working.
- The trench will pass partially through the edge of drainage detention basin RR2 adjacent to W Belmont Avenue. The incursion of the HST route into the basin will result in a small reduction in the capacity of the basin. Refer to the Final FEIR/EIS for the Merced to Fresno Section for the mitigation requirements in this area.
  The PMT gave direction that in this area the shoring wall should be specified as a permanent rigid wall structure. This is required to act as a first line of defense, protecting the U-trough in the event of disturbance to the basin slopes and providing additional lateral restraint to the U-trough.

- Also in basin RR2 near W Belmont Avenue, the existing storm drainage system outfalls into the basin via a 96-inch-diameter pipe that crosses the proposed HST route. The pipe must be diverted because its current invert is at a level that would conflict with the U-trough. The diversion route is shown on the utilities drawings and the structures drawings where it runs alongside the U-trough for approximately 500 feet. The timing of its removal and construction of the diversion will be a critical aspect for coordination of construction and scheduling of the works in this area.
- The SJVR departs from the UPRR at two points. Both spur tracks cross the proposed route of the HST. To allow for this, short sections of covered trench have been designed. In order to maintain operational usage of the SJVR during construction, these sections of trench should be constructed at different times. Constructing the southern crossing first may ease the accessibility for construction of the northern crossing.
- Dry Creek Canal crosses the proposed route close to the location that the southern SJVR spur also crosses Dry Creek Canal. Consequently, the existing bridge that carries the SJVR over Dry Creek Canal must be removed. In this area, the trench has been designed as a cut-and-cover structure.

  To provide clear separation of responsibility and ownership between the HST trough structure and the canal structure, a box culvert has been designed to cross over the HST U-trough. To ensure that the structures are separate, 1 foot of earth fill should be placed over the U-trough slab and below the base of the culvert.

  The culvert concept is a simple 2-cell RC box structure. A 2-cell structure has been selected to minimize the thickness of the top slab and therefore limit the amount of necessary vertical realignment to the SJVR while maintaining the existing soffit level.

  Initial discussions with the owners of the canal (Fresno Irrigation District) have confirmed that the concept would be broadly acceptable, subject to providing the ability to block off the cells for maintenance individually and with headwall details that match the profile of the existing canal on either end of the structure.
- The HST trench crosses under SR 180 at a point where it is on embankment. The alignment of the trench also conflicts in plan with the abutment of a bridge that takes the SR 180 over the UPRR and H Street. To avoid major disruption of SR 180, it is proposed that this section of the trench should be constructed using a box jacking technique.

2.3 Construction Methods Assessment

2.3.1 Main Trench

The design team considered construction of the Fresno Grade Separation in the Constructability Memo at the 15% design stage. The preliminary design has undertaken outline calculations based on assumed construction sequence to demonstrate the adequacy of the shoring system.

2.3.1.1 General Trough Excavation

The basic construction sequence described in section 2.2 and shown in Table 2.2-1 is extended slightly for the covered sections as follows:

- Construct temporary shoring walls
- Excavate to formation level, inserting temporary props as required
- Construct U-trough base slab
- Incrementally construct the side walls, removing temporary props as encountered and constructing permanent props if required by the design; where the section is covered, construct the roof lab using falsework supported from the base slab
- Backfill over the covered sections

The above sequence could be applied over the full length of the trench where the U-trough is used, or it could be implemented in discontinuous sections if routes are available for removal of excavated materials. Because access to the excavation is difficult in the middle sections, it is likely that the contractor will choose to excavate the full length of the trench prior to constructing the U-trough, except at high-risk locations.

The high-risk locations are likely to be Belmont Basin, Dry Creek Canal, and SR 180. In these locations, the design team believes that a contractor would choose to do local excavations early in the construction period to overcome accessibility problems for the remainder of the trough construction.

2.3.1.2 Roeding Park Area

At the section of U-Trough adjacent to Roeding Park, the diversion of the 96” storm drain runs parallel to the trench for approximately 500 feet. Over this length the invert level of the storm drain rises relative to the U-Trough such that the pipe will lie alongside the U-Trough. As the width available for construction of the U-Trough and the Storm Drain is restricted, it may be necessary to vary the expected construction sequence either to install the drain and U-Trough within a shoring wall on the boundary of Roeding Park or to construct the storm drain in advance of the U-Trough and then install the shoring wall alongside the storm drain.

2.3.1.3 E Belmont Avenue

The existing East Belmont Avenue will be closed temporarily in order to construct a new overcrossing bridge structure. Once the overcrossing is constructed, the road would be re-opened and the U-Trough structure constructed beneath it. However, when the bridge beams for the overcrossing are installed there may be insufficient vertical clearance for normal piling equipment, in which case low height equipment may be necessary. However, if the shoring wall can be constructed before the beams are installed this constraint can be avoided.

2.3.2 Jacked Box Concept and Constructability

Box and structure jacking has been used in many parts of the world over the last 50 years. It has become a well-established and successful technique in that time. In practice, there are many different forms and methods of jacking that can be used. Many of the techniques used are covered by patents, and as a result, it is likely that the successful DB contractor will employ a specialist subcontractor for this work who uses only one technique.

2.3.2.1 Description of the Structure

The jacked box is to be situated beneath the existing State Route 180 overpass. The overpass consists of two 3-span bridges accommodating the eastbound and westbound traffic. Both bridges are formed with a combination of precast trapezoidal box girders for span 1 and cast-in-place box girders for spans 2 and 3. In both instances, the RC box girders have been prestressed. The western abutments, nearest to the jacked box location, consist of RC pad footings, RC stems and bearing seats, and RC wingwalls. At its nearest point, the top of the jacked box is situated approximately 8 feet below the soffit of the western abutment.

Where the jacked box is to be constructed, the proposed right-of-way has been increased to 80 feet because the excavation shoring walls would be constructed farther apart than in the other parts of the excavation to allow sufficient working space for construction of the box. As the excavation must be unbraced to allow space for constructing the box, it is likely that the shoring walls will also be more substantial in this area than in other parts of the U-trough. It is expected that the contractor will wish to extend the shoring to permit the construction of overhead braces that clear the top of the box (due to the topography of the area, the top of the box projects above ground level in the launch pit).

The box would be constructed on a base slab that is used to provide the reaction force against the jacks. This "jacking base slab" is also likely to be dowelled to the shoring wall to further distribute the jacking forces. Depending on the method used by the contractor, it is also possible that the jacking base slab will be extended part way up the sides of the jacked box as a way of providing lateral guidance to the box to ensure it stays properly aligned in the early (critical) stages of the jacking operation.

The box has been assumed to be a monolithic RC section, though it is also possible that the contractor may choose to divide the box into segments with "interstage" jacking between segments.

The internal dimensions have been selected to satisfy clearance requirements; however, the box cross section must achieve the "free space" (aural passenger comfort) requirement for high-speed train operation. The length of the structure is to cover the whole Caltrans right-of way.

The preliminary design has assumed the following key dimensions for the jacked box:

- Length (excluding shield): 240 feet
- Thickness of the walls, roof, and base: 5 feet
- External width of the box: 53 feet
- External height of the box: 42 feet

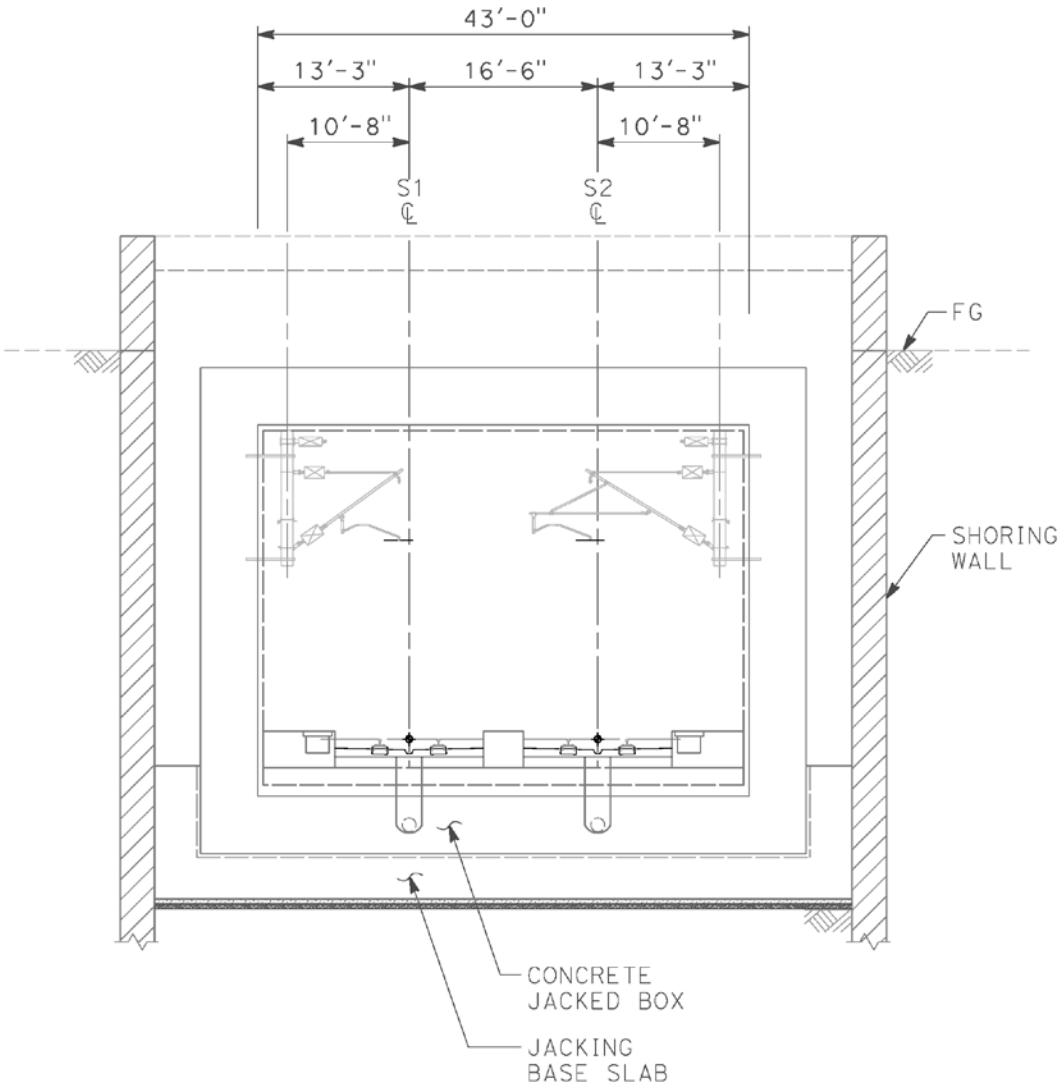


Figure 2.3-1
Cross Section of Launch Pit with Box in Position

A view showing a similar-sized box structure under construction in this situation is shown in Figure 2.3-2.

Figure 2.3-2
Example of a Partially Constructed Box in Trench Prior to Jacking

At the leading edge of the box, a purpose-designed tunnel shield would be cast on to the normal wall of the box. This will incorporate a steel cutting edge that may also be adjustable as a method of steering the box during jacking. At the rear of the box, additional fixtures may be added to accommodate the thrust jacks. A typical cutting edge is shown in Figure 2.3-3.

Figure 2.3-3
View of Shield and Cutting Edge

Note: In this case, there is no roof slab as the excavation will be open-topped.

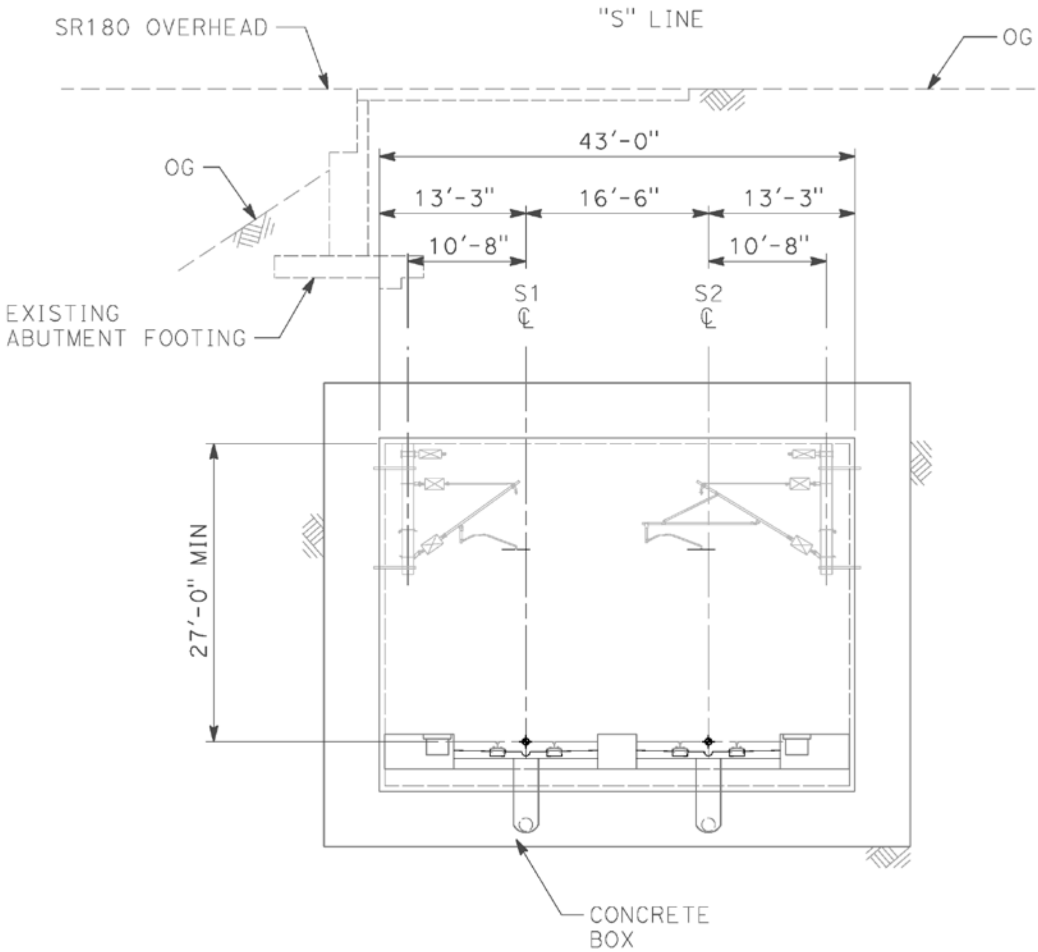


Figure 2.3-4
Cross Section and Key Dimensions of Jacked Box with Indicative Relationship to SR 180 Bridge Abutment

2.3.3 Anti-Drag System for Box Jacking

An essential component of the box jacking system is the method by which drag from the structure is reduced. This is required because as the box is jacked forward, there is a tendency for the box to drag the overlying ground along with it. In large embankments, there is some resistance to the drag force from the shear resistance of the embankment itself. However, this resistance may be insufficient to restrain the effect in the case of a wide box with low cover. If unrestrained, the ground on top of the box would be dragged forward, causing major disturbance and possible disruption to the overlying infrastructure.

The anti-drag system (ADS) is designed to prevent this — its use makes it feasible to consider box jacking where the depth of cover is as low as 6 feet. The action of an ADS is illustrated in Figure 2.3-5.

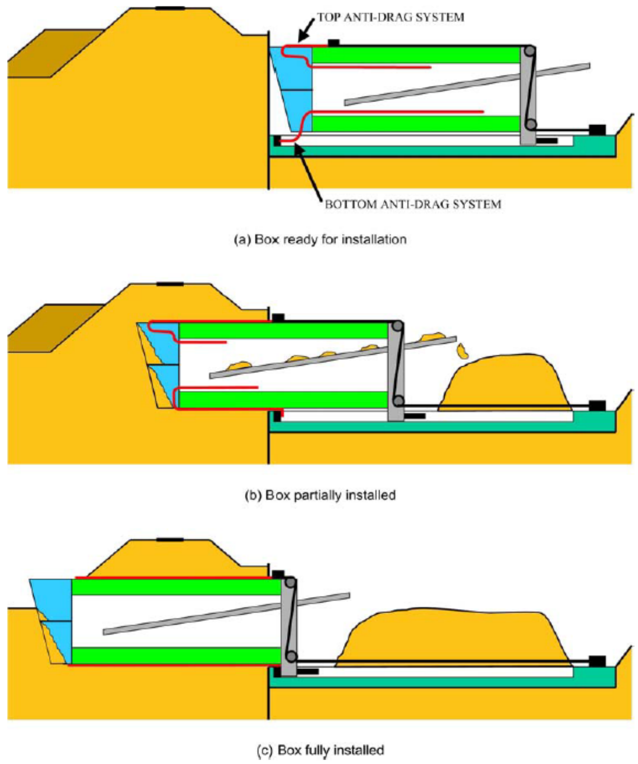


Figure 2.3-5
Illustration of Use of Anti-Drag System in Excavation

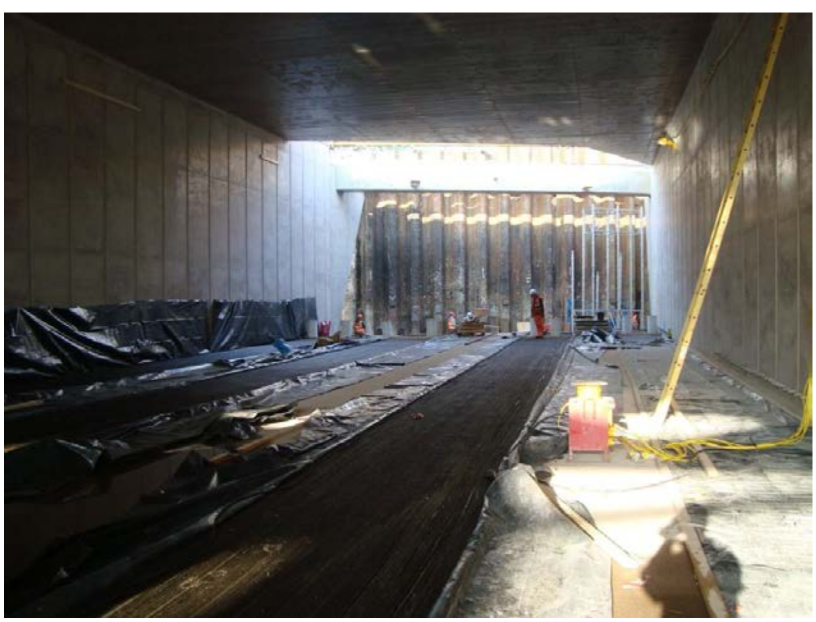


Figure 2.3-6
Anti-Drag Cables Laid Out Prior to Commencement of Jacking

One proprietary ADS comprises arrays of closely spaced greased wire ropes. The lower ADS wires are anchored to the jacking base, with their free ends passed through guide holes in the shield and stored with their free ends inside the box (the lower red line in Figure 2.3-5). As the box advances, the ropes are progressively drawn out through the guide holes in the shield and form a stationary (anchored) layer between the moving box and the ground below. The jacking forces are absorbed by the ADS and transferred back into the jacking base by the wires.

The upper ADS wires are anchored to a frame above the box with their free ends passed through guide holes in the shield and stored inside the box (the upper red line in Figure 2.3-5). As the box advances, the wires are drawn out through the guide holes to form a stationary layer that is anchored to the frame and isolates the ground above the structure from the jacking force. The wires transmit the jacking force back to the anchor frame.

In this manner the ground above and below the box is isolated from the drag forces and remains largely undisturbed.

Other systems that provide anti-drag capability follow the same basic principle but may substitute steel strips for the wires described here or use scrap conveyor belting to fulfill the same function.

The ADS wires do not isolate the sides of the box from the jacking force, so it is necessary to provide a method of reducing the frictional resistance of the sides to ensure that the force transmitted to the ground at the sides is minimized. Ground drag on the sides of the box is usually reduced by arranging the cutting edge so that a slightly larger hole is excavated than the box dimensions. Typically, the excavation is oversized by about 1 inch. However, the amount of over excavation has an effect on the amount of settlement that is seen at the surface, so overdig should be kept to the minimum necessary. Ground drag can also be reduced by lubricating the ground/structure interface with bentonite slurry. Usually both these methods are used together.

To provide lubrication, slurry injection tubes would be cast into the walls of the box during construction. These tubes would be connected to a master valve linked to the bentonite supply pipe. Figure 2.3-7 shows a set of bentonite injectors arranged in a wall that is ready for concreting.

Figure 2.3-7
Bentonite Slurry Injection Tubes

On completion of the jack, the bentonite injection tubes would be filled with cement grout to make a permanent seal.

The arrangement described above (with the exception of the upper ADS wires) is shown in Figure 2.3-8, which was taken at a recent railway project in the United Kingdom.

Figure 2.3-8
View of Jacking Area with Anti-Drag Restraints at Bottom

2.3.4 Methodology for Jacking

The design team has developed a methodology to jack the box into place. This methodology has been discussed with a specialist jacking contractor, who has commented on the methodology and confirmed that it is feasible.

This methodology is as follows:

- Prior to construction of the jacked box, construct a structural base slab. This slab is designed to guide the box during jacking and provide a reaction base against which the jacking force can be applied.
- Lay a lubricated layer of sheeting on the jacking slab. This sheeting and lubricant could be a variety of materials, but the contractor consulted preferred steel plates as sheeting because less rigid materials have a tendency to ripple and jam the jacks. The concrete box section will be constructed on this layer.
- Construct the concrete box using normal RC techniques. Because of the need to construct the box in the bottom of the trench, it may be difficult to prop the shoring walls in this area. Surveyed ground levels indicate that the roof of the box will be above ground level in this location. Consequently, the design has assumed that a more substantial shoring wall section that requires no bracing within the height of the box would be used. It is also possible that the shoring wall could be extended to a higher level so that bracing could pass over the box.
- Prior to commencing jacking, it may be necessary to undertake ground improvement to the embankment fill that the box will pass through and beneath the bridge abutment to ensure that settlements of the SR 180 abutment remain within specified limits.
  Grouting may be necessary only to ensure that the excavation face is stable and provides enough support to the upper layers of the embankment.

Grouting may need to be more extensive to limit the amount of settlement experienced at the surface if the embankment materials are particularly sensitive to disturbance. The amount of ground improvement needed depends on the settlement tolerance specified.
It may also be necessary to use a multicellular face shield so that the excavation face is limited to smaller pockets that can be supported individually and excavated independently.

- Once constructed, jack the box against the shoring wall that closes the end of the trench; this is expected to be broken out from within the box. Apply the jacking force through jacks reacting against the jacking slab at the rear of the box. Concrete spacer blocks may be used to adjust the jack location as work proceeds. Additional lateral support to the shoring wall will be required to ensure stability after cutting off the lower part of the shoring wall within the box.
- Continue jacking the box and excavating the face from within the box. The jacking force may be reduced by the injection of bentonite or other lubricants between the outer face of the walls and the ground as work proceeds.
- On completion of the jacking, the cutting edge and face shield will be broken out to a point where they can be incorporated into the permanent trench walls.
- Decommission the jacking pit and complete trench construction by constructing a standard trench cross section within it.
- Backfill the space between the temporary shoring wall and the finished U-trough.

2.3.5 Support for the Excavated Face

Excavation of jacked boxes of this nature requires a balance between the rate of excavation of the material that the box is passing through and the rate at which the jacking force advances the box into the material. A secondary concern is that the excavation face will collapse in an uncontrolled way leading to over-break at the edges of the box. This may lead to the migration of material from outside the excavation zone into the excavation, which eventually results in excessive settlement at the surface. In the worst case, this might result in collapse of the overburden materials into the excavation (see Figure 2.3-9).

The above sequence is more likely to occur in loose granular materials than in stiff cohesive materials. The SR 180 embankment is assumed to be constructed of well-compacted granular materials similar in nature to the in situ ground. This reduces the risk of collapse of the face.

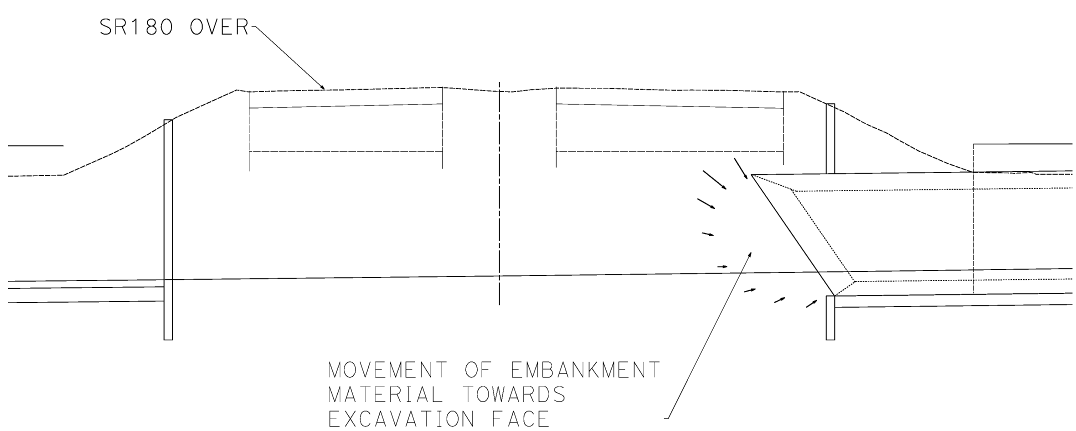


Figure 2.3-9
Excavation Process during Jacking

There are a number of ways to mitigate the risk of face collapse:

- **Pre-excavation grouting**
The use of either chemical or cementitious grouts to increase the adhesion between the soil particles so that the excavated face behaves as a uniform, stiff, self-supporting mass during

excavation. Grouting can be done either from the surface along the line of the jack or at intervals during excavation from the excavated face.

- **Compartmented excavation faces with support panels**
  The excavation size proposed for this box is approximately 53 feet wide by 42 feet high. This would present a large excavation face that may be difficult to control. In poor ground, it is common practice to subdivide the excavation face into several compartments that can be excavated by mini-excavator or by hand. This method gives the ability to control the excavation by selectively excavating certain compartments at different rates in order to steer the box and maintain directional control. Some contractors also use doors that retain the face when not being excavated. These may be hydraulically controlled and linked to the main jacks to ensure a constant pressure is exerted on the face.
- **Ground freezing**
  As an alternative to chemical or cementitious grouting, ground freezing increases the uniformity and cohesion of the excavated face by using the intergranular groundwater to bind the soil particles together for excavation. This technique is most commonly used where the excavation is below groundwater level so there is an abundant supply of water. However, because the freezing of water is an expansive process, this also means that there is a risk of heave at the surface. In extreme cases, the frozen mass can become marginally buoyant, leading to substantially increased heave.

Of the above techniques, ground freezing is considered inappropriate, as there is unlikely to be sufficient groundwater present for it to be effective.

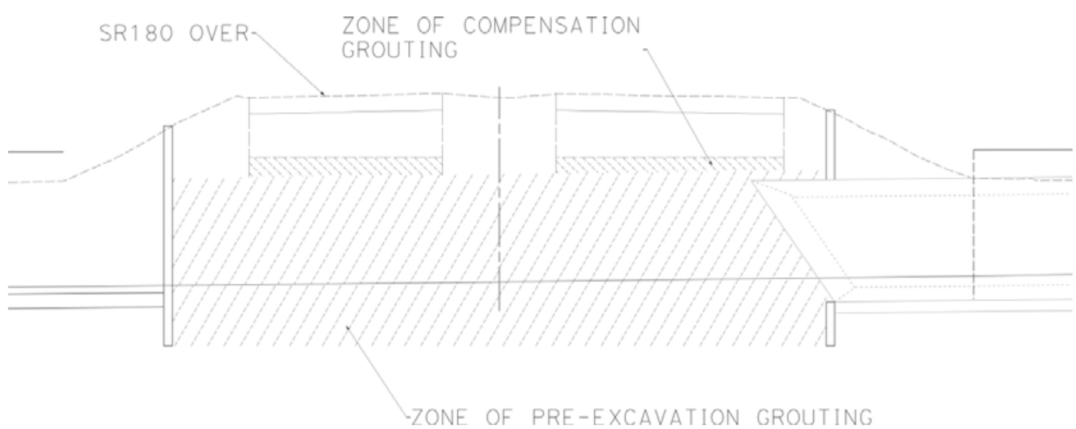


Figure 2.3-10
Zones Where Pre-Excavation Ground Treatment and Compensation Grouting May Be Used

The design team believes that the contractor will choose to use a combination of general pre-excavation grouting, compartmented excavation, and grouting from inside the box in advance of the excavated face. This combination works together quite conveniently — once excavation has started, it is possible for grouting to be done from one compartment while excavation is underway in other compartments. This also means that the excavation process can be regarded as a continuous operation. This is important as a major factor in maintaining the stability of the face comes from setting up a uniform “flow” of material through the box. If the process had to be stop/start with large time intervals between, it would be more likely to allow local collapse of weak areas, which would disrupt the uniformity of the “flow” with unpredictable results.

In cases where face collapse becomes a problem, the seemingly counterintuitive solution is often to increase the rate of excavation. This means that the calculation of required jacking force should be conservative to ensure substantial additional capacity is available if needed. In soft ground, a cellular

shield configuration is normally adopted with the internal walls and decks buttressing the tunnel face. A cutting edge around the perimeter of the shield accurately cuts the hole through which the shield body and box structure pass. These cutting edges are sometimes adjustable to assist in the steering of the box. The shield provides safe access to the tunnel face for miners and machine operators, and egress for the ADSs.

The ground must have sufficient strength to arch safely across the open cells and must accept the incremental advance of the shield into it without distress. Sometimes it is necessary to improve the ground in advance of tunneling. In the ground conditions expected at the site, grouting ahead of the excavation is recommended if the water table is confirmed below the excavated profile.

Typically, 0.5 feet of soil would be trimmed from the face, and then the box would be jacked forward 0.5 feet. This sequence is repeated until the tunneling operation is complete, thus maintaining the necessary support to the face.

2.3.6 Calculation of Jacking Load

The jacking load will consist of the following:

- Reaction on shield structure
- Friction due to the dead load of the concrete structure on the ground/concrete launch portal
- Friction on the top and side of the concrete box against the soil

2.3.6.1 Reaction on Shield Structure

The reaction on the shield structure is assumed to be the passive pressure from the cutting edge of the shield. The thickness of the cutting edge is usually used to determine the reaction, and the resistance is calculated as the passive reaction on that area.

Based on experience on other projects, a 2-inch cutting edge around the perimeter has been assumed. Conservatively the outside perimeter is used.

Total Area: 2in x (2 x (516” + 624”)) = 4,560 in^2

The passive reaction is calculated at the mid-level of the box (265-feet above datum):

$$\sigma_P = K_P\, z\, g = 4.71 \times (320 - 265) \times 125 = 225 \text{ psi}$$

$$F = 225 \times 4560 = 1{,}026 \text{ kips}$$

A factor of safety should be applied. It is suggested to use 3.0.

$$F = 3{,}042 \text{ kips}$$

2.3.6.2 Friction Due to Dead Load

The design of the component force for the weight of the structure is based on the dead load of the structure. This should be multiplied by a suitable coefficient of friction. The coefficient of friction between concrete and steel is conservatively assumed to be 0.3.

This is an upper bound value as both within the box and within the excavated profiles, the ADS formed by a series of greased wires at the interface between the top and the bottom surface (where the largest loads from gravity are expected) will drastically reduce this contribution.

Self-weight:

$$230\text{ft} \times ((42 \times 53) - (32 \times 43))\ \text{ft}^2 \times 156\text{pcf} = 30{,}498 \text{ kips}$$

Frictional force (FF) assuming a friction coefficient of 0.3:

FF = 30,498 kips x 0.3 = 9,149 kips

To include weight of shield and a factor of safety, the dead load should be multiplied by 1.2. Therefore the frictional forces would be 10,980 kips.

2.3.6.3 Friction from Soil and ADS

It is expected that the use of the ADS will impose additional forces in the jacking system. Based on references of similar projects, it is expected this load will be less than 4,500 kips.

The total load, including all the components defined above, would therefore be 18,500 kips. The maximum design single jack load is assumed to be 500 tons (1000 kips), and the number of jacks is expected to be less than 20 units. The jacking pit and the portal structure have been verified for this load.

2.3.6.4 Ground Control

As discussed previously, the soft ground will most likely need to be pretreated to provide sufficient stand-up time during tunneling. In addition, the ground may need to be stabilized in advance to control surface settlement when tunnel jacking at such a shallow depth.

2.3.6.5 Monitoring

The jacked box tunneling operation must be carefully monitored and controlled to ensure the required performance and safety. Throughout the tunneling operation, movements at the ground surface over the area affected by the tunneling operation, jacking forces, and vertical and horizontal box alignment should be regularly monitored and compared to predicted or specified values.

Caltrans also requires monitoring of the bridge wall.

2.3.6.6 Ground Settlement

The ground movements, including settlement due to the jacking of a box, are highly dependent on the method of construction, shield design, ADS's, and preparatory works. Most of the key parameters depend on the choice of temporary works, so the temporary works contractor would normally carry out the settlement assessment.

The settlement limits stated by Caltrans (see 2.3.9) relating to the abutment of the SR 180 Bridge are onerous. It is considered likely that the contractor would need to implement a compensation grouting system that will inject grout into the area below the foundation of the abutment in order to maintain or restore its original position.

In some compensation grouting schemes, the grout injection system may be linked to the movement monitoring system to automatically inject grout when the movement exceeds some defined threshold.

2.3.7 Alternative Methods of Constructing the HST Route Under SR 180

During the 15% stage of design development a number of alternative methods of constructing the HST trough in this location were studied.

These fell into two categories:

- Working under the SR 180 while in use
    - Using a jacked box
    - Propping the superstructure and using temporary bridges to carry traffic while excavating beneath to construct the U-trough

   - Extending the SR 180 bridge by adding a further span before excavating the U-trough below and through the extra span

- Closing the SR 180 in some way
   - Closing one travelway while running both directions on the other
   - Implementing full closure with diversion routes

Of these alternatives, the use of the box jacking technique was thought to be the least disruptive to Caltrans operations.

2.3.8 Summary of Feasibility Design

The 15% design assumed that the box would be constructed in the U-trough to the south side of the SR 180 embankment. This site is a building to be demolished, and consequently there is an area of land with easy road access that may be used for temporary construction. There is no equivalent to the north of the SR 180.

The procurement design has developed the requirements for jacking a box and has confirmed the following:

- A structural design for the box can be achieved that also allows for the loads from the SR 180 bridge above
- There appears to be adequate clearance between the jacked box and the SR 180 bridge foundations (based on interpretation of the as-constructed drawings)
- The jacking force required to propel the box is achievable and in keeping with that required for similar structures on other contracts
- An experienced box jacking contractor considers the proposed method achievable
- There are ground treatment techniques that would render the embankment material suitable for the controlled excavation needed for the proposed technique

2.3.9 Discussions with Caltrans about the SR 180 Bridge

The design team met with Caltrans on October 23, 2011, to discuss the proposals for the box jacking and to determine their requirements for the following:

- Control of settlement of the SR 180 structure during the box jacking process
- Reinstatement of the bridge afterward should this be necessary

The team explained that the box would pass directly below the abutment foundation of the SR 180 bridge, and information was requested relating to permitted settlement of the structure.

Caltrans subsequently provided information that can be summarized as:

- The abutment movements must not exceed ¼ inches horizontally ½ inches vertically, whereas
- The vertical deck movement must not exceed 1 inch for continuous superstructures and 2 inches for simple spans
- All proposals relating to crossing of the SR 180 will be subject to Caltrans review and approval before work is permitted to commence

In order to comply with these movement limitations it is likely that the contractor will be required to undertake extensive grouting of the ground under the abutment. It may also be necessary to install compensation-grouting equipment linked to a settlement monitoring system to adjust the foundation of the bridge as jacking proceeds.

2.4 Temporary Construction Loadings Considered

During the construction of the U-trough, a number of temporary construction loads will be present for short or long periods. Refer to TM 2.3.2 clause 6.4.4.

The shoring design allows for the following:

- The effect of a Cooper E80 Train set on the Union Pacific tracks adjacent to the excavation. The peak pressure of 1882psf at underside of tie level has been converted into an equivalent uniform surcharge load of 420psf applied at ground level adjacent to the wall. (See 2.2.1.)
- A surcharge pressure of 600psf has been applied to areas where construction activity may use land adjacent to the U-trough. This is not additional to the train loading above and is also not applied in areas where construction access is not permitted.
- Variable groundwater levels in the section of the trench adjacent to the Belmont Basin and in the area of the Dry Creek Canal crossing.

2.5 Temporary Construction Easements

Temporary construction easements are required for the construction of the following:

- The diverted 96-inch storm drain outfall
- Dry Creek Canal structure
- SJVR connections
- Connections to the trench drainage sump
- Emergency egress stairwells and emergency access roads

The drainage sump is located between two spur tracks and will be connected to the local drainage system via a new detention basin. The basin will be constructed adjacent to the southern SJVR spur line.

2.6 Traffic or Pedestrian Diversion and Control

The construction of the trench requires the permanent closure of W Belmont Avenue Underpass, N Thorn Avenue, and part of Golden State Boulevard. Replacement overcrossing bridges are to be provided at W Olive Street and W Belmont Avenue.

Traffic management will be necessary to accomplish these changes. The contractor will be required to coordinate and plan works in these areas so that traffic disruption is minimized to the satisfaction of the City of Fresno.

The following mitigation measure has been identified in section 3.2.7 of the Merced to Fresno Final EIR/EIS and could be utilized during construction:

- W Belmont Ave: install traffic signals at the intersections to improve LOS and operation

For the construction of the U-trough, there will be a need for construction entry and egress points that connect to the road system. It is expected that the majority of excavated material from the U-trough will need to be taken offsite via these egress points, so it will be necessary to agree upon the amount, frequency, and operating hours for these entry/egress points with the City of Fresno.

2.7 Drainage Concept

The track drainage within the trench will be carried in two longitudinal pipes cast into the base slab in accordance with the directive drawings. At the low point of the U-trough (STA 10926+00), the drainage flow will be collected at a sump adjacent to the west side of the trench structure where it will be pumped to a new detention basin located within the environmental footprint adjacent to the southern SJVR spur

line. Outfall from the basin will be attenuated to discharge only at the rate of a 2-year storm as discussed and agreed with the Fresno Metropolitan Flood Control District (FMFCD).

For design, it has been assumed that the depth to the natural groundwater level is around 60 feet below ground level, except in areas where higher or perched water levels may be expected. This assumption is based on historic borehole data from Caltrans projects in the area in the absence of more recent information.

Higher groundwater levels have been assumed to exist at the Drainage Detention Basin (RR2) adjacent to W Belmont Avenue and at the point where Dry Creek Canal crosses the HST route. In both cases groundwater has been assumed to be 10 feet below ground level as recommended in the Geotechnical Design Basis Memorandum (Appendix A).

A ground investigation has been commissioned, but it will not be able to provide improved data before completion of the procurement design phase.

Based on the above assumption, it is not expected that cutoff walls will be required at the ends of the trench to limit groundwater inflow. Buoyancy checks have been carried out assuming groundwater levels as above. These checks show that any additional measures to counteract buoyancy are not required.

2.8 Emergency Egress and Escape Provision

Although not strictly an elevated or underground facility, the team has agreed that it is appropriate to apply the requirements of NFPA130 for emergency escape/egress to the U-trough. This means that escape stairwells are to be provided at maximum 2,500-foot intervals through the box. Stairwells are provided as indicated in Table 2.8-1.

Table 2.8-1
Stairwell Provisions

STA	Locale	Egress features
10906+00	Adjacent to communication site, located in the abandoned connection of Golden State Boulevard to W Belmont Avenue	Stairwell is located close to the communication site and will share a common road access track. There is space for provision of a turning area for vehicles.
10925+00	Between the north and south SJVR spur connections	Emergency services access to the location of the stairwell will need to be agreed with the owner of the facility. There is space for provision of a turning area for vehicles.
10950+00	South of Divisadero Street and adjacent to G Street	Stairwell is located in an area currently used as a vehicle parking area with a frontage onto G Street. There is space for provision of a turning area for vehicles.

Each stairwell is 10 feet wide by 25 feet long to allow for the later installation of a staircase.

The staircase is assumed to be 44 inches minimum width with 5-foot-wide landings at 12-foot vertical intervals and 21 treads per flight.

2.9 Inspection, Service, and Maintenance Access

The trench structure itself will be a simple massive RC structure with a limited number of movement joints at intervals. There will be no specific provision for inspection or maintenance access other than the general maintenance access to the route.

The drainage sump will require pedestrian access at the surface and access for the installation and removal of pumps. Pedestrian access will also be provided by construction of an access door from the emergency walkway within the trench. Providing this door increases the risk that it may be dislodged by the passage of a train, so it is proposed that this door and the doors associated with the emergency escape stairs should be sliding doors. These may be fitted during a later contract.

Access for pump replacements will require a permanent easement and is likely to be via the area of land between the SJVR spur tracks.

Movement joints in the walls will be required to limit the effects of temperature and ground movement. These joints are intended to be no more complex than simple cast-in waterstop details.

2.10 Utilities Affected and Disposition

A number of existing utilities cross the route of the trench or are within the proposed right-of-way. Where these can be diverted, the proposed diversion route has been identified on the utilities and structures layout drawings. It has been a principle of this work to divert utilities into new infrastructure (such as road overcrossings) or into the fill over the covered parts of the trench where possible. Where there are specific crossing points that cannot be accommodated in this way, a utilities crossing structure is incorporated into the detail of the trench or the trench design has been modified to accommodate the utility.

Examples of where the trench design may be affected are as follows:

- **Kinder Morgan hydrocarbon line**
  This utility does not in fact enter the proposed right-of-way of the HST route. It runs along the UPRR right-of-way in an easement granted by UPRR. Its precise route varies along the right-of-way and in some places appears to be within 5 feet of the right-of-way. The location shown on the utilities plans is based on information provided by Kinder Morgan, but its accuracy has not been verified by excavation.
  For the construction of the trench, care must be taken to consider the effects that the trench construction methodology will have on this utility. At this time, all that is known about the line is that its diameter is 12in.
  Concern is based on the following:
    - The pipeline has been in service for around 30 years, and its current condition is unknown to the design team.
    - Given the above, it is unknown whether the pipeline is sensitive to the magnitude of ground movement that may be expected from construction of the U-trough.
    - The pressures at which hydrocarbon lines operate are usually very high in order to minimize the number of intermediate booster stations required. Consequently, a break in the line could occur explosively and be difficult to contain.
    - The line is reported to be buried deep enough to pass under the depressed Fresno Street, which suggests it may be up to 20 feet deep. This depth is a further indication that the operating pressure of the line is high.

- o The design team does not know whether the pipeline is currently leaking into the surrounding ground or has leaked in the past. The presence of hydrocarbons in the excavation would influence the choice of excavation methods that a contractor would use in the U-trough excavation.

In order to clarify these issues Kinder Morgan were contacted and provided an initial response by e-mail on December 5, 2011.
The main points are:

- o *Location and Depth: Exact Location & depth and only be determined by potholing. The Alignment sheets will give the general location but no depth information.*
- o *Type of pipe & diameter: this information is on the alignment sheets generally speaking, it is steel pipe 12.75" OD. Wall thickness varies.*
- o *Foundation beneath the pipe not sure exactly what they are asking, usually the pipe is bedded in clean soil.*
- o *Contents and pressure within the pipe: liquid petroleum products (motor and jet fuels). The maximum operating pressure (MOP) is around 1440 psig; however, the operating and control pressures will vary along the pipeline.*
- o *Condition of the Pipe: the pipe meets or exceeds all regulatory requirements.*
- o *Date of last inspection: KM has a robust inspection program; however, I do not see how this information is pertinent to your design team.*
- o *General performance: overall good.*
- o *Allowable movements: Lets discuss at our meeting, I need to know the context and purpose of movement.*
- o *Design criteria: 49 CFR 195*
- o *Support Methodology & serviceability criteria of the support: KM will determine the adequacy of any proposed supports.*
- o *Local soil lithology: I don't believe we have the information for the hundreds of miles of pipelines that KM operates in the State.*

This information confirms that the working pressure of the pipeline is likely to be high.
No clarity is provided as yet regarding tolerance to movements of the ground or proximity to the shoring walls.
It is not clear how the pipe can be protected from ground movement but it is known that the pipe is placed in the earth of the trench. So that there are no additional elements that may stiffen the pipes response to movement.

Overall the pipeline’s proximity to the excavation remains a concern. Therefore, it is recommended that this pipeline be diverted to the east side of the UPRR right-of-way prior to construction of the U-trough structure.

- **96-inch storm drain outfall crossing the HST route at STA 10897+30**
  This is a diversion of the existing outfall to drainage detention basin RR2 that is located adjacent to W Belmont Avenue. The diversion of the existing facility is essential to the construction of the U-trough in this area and the diversion route that is indicated on the drawings lies in close proximity to the trench.
  After crossing under the HST route, the storm drain outfall runs parallel to the U-trough for over 500 feet until it reaches the detention basin. Its location, between Roeding Park and the U-trough, will be a substantial constraint on the working space available for construction of both the U-trough and the diversion. Both must therefore be considered together when developing the methodology for construction in this area.
  The vertical position of the storm drain is also a constraint in the location of the emergency escape stairwell.

- **12-inch water line at STA 10915+60**
  This is a small-diameter water supply pipe whose route cannot avoid crossing the U-trough. The diverted utility passes around the edge of the detention basin before crossing the HST via a utility crossing bridge. The utility crossing bridge will be a concrete box that will totally enclose the sleeve through which the utility pipe is installed. Future maintenance of the utility will be carried out by withdrawing the pipe from its sleeve.
  At this location there are also a number of gas lines that cross the U-Trough so that the utility crossing structure is likely to be around 10-feet in width.
- **30-inch sewer line at STA 10933+30**
  This is an existing gravity sewer that currently passes under Dry Creek Canal. The diversion crosses the route at a vertical clearance of 24 feet because a pumped solution is considered unacceptable by its owners.
- **Dry Creek Canal culvert at STA 10934+05**
  Dry Creek Canal will be culverted to pass over the trench on its current alignment and invert level. In order to maintain separation of the culvert structure from the trench structure, a minimum thickness of 1 foot of fill is to be placed between the upper surface of the cover slab and the culvert foundation.
- **12-inch Gas Line**
  This pipeline is a diversion of an existing line and passes through the fill covering the HST adjacent to the Southern SJVR spur.
- **60-inch storm drain diversion at STA 10935+85**
  This is a new storm drain that is the diversion route for a drain that currently crosses the route of the HST at Divisadero Street. It is a gravity design and crosses the U-trough in a concrete sleeve structure at a minimum vertical clearance of 24 feet.
- **SR 180 route crossing at STA 10937+00 to 10939+50**
  It is believed that any existing utilities along the route corridor are relatively shallow. The U-trough adjacent to the SR 180 is at considerable depth. The design concept in this location is for a large concrete box to be jacked through the embankment of SR 180, passing underneath any near surface utilities and the SR 180 bridge abutment at a depth of approximately 20 to 30 feet below road surface. The utility plans indicate an abandoned oil pipeline that, from its alignment, predates the construction of the SR 180 embankment. The utility information does not indicate that the pipeline was removed during construction of the embankment, so it is assumed still present. The depth of other oil pipelines in the area suggests that this line is at approximately 10 to 15 feet below ground level, which means that it would be encountered during the excavation of the jacked box. The contractor will need to be prepared to deal with the excavation of potentially contaminated ground on the route of the pipeline.
- **20-inch water pipe at STA 10940+15**
  Similar to the other crossing, this is a service line that cannot avoid crossing the route of the HST and for which there is no reasonable alternative route. The pipe will be carried by a concrete surround and will be sleeved through the structure to permit removal and replacement.
- **Flood overflow at STA 10942+80**
  This is not strictly a utility and the purpose of this structure is discussed under hydrological issues in the next section.

2.11 Hydrological Issues

These issues are discussed in detail in the Floodplain Impact Assessment Report.

The main impact of the trench design is to ensure that the trench wall is substantially higher than the 100-year flood level in the Dry Creek Canal area. In this area, the 100-year flood level is approximately at ground level. Protection against flooding will be provided indirectly because the requirements for collision/intrusion protection require a wall 10 feet higher than ground level and at the west side the trench wall is 3 feet high.

During discussions with FMFCD, it was noted that the area to the south of SR 180, north of Divisadero Street, and to the east of the HST route would be cut off by the construction of the U-trough. The FMFCD has commented that in extreme flood events (50-year return period or more) this area can develop an overland flow toward the west that relieves flooding to the east. The FMFCD would like this "relief valve" to remain after construction of the U-trough. To provide for this, a closed box (similar to a utility crossing) has been added to the trench approximately at ground level. Under normal circumstances, this structure will be completely empty, but in the extreme cases described, it will allow water to flow across the HST route.

2.12 Noise Mitigation and Acoustic Treatment

The Merced to Fresno Final EIR/EIS provides information on operational noise mitigation requirements that have been adopted by the Authority. Implementation of operational noise mitigation is not part of the scope of this design build contract. However, project facilities that will be completed under this contract must be designed to accommodate future noise mitigation elements.

2.13 Details of the Geotechnical Parameters Used for Design

The geotechnical parameters are described in the Geotechnical Design Memorandum attached at Appendix A.

3.0 The Jensen Trench

The Jensen trench is an RC U-trough structure similar to the Fresno Grade Separation. It varies in depth from approximately 0 to 15 feet. The primary reason for the trench is to allow the HST to pass under the E Jensen Avenue road bridge and to protect the HST from the effects of flooding from the FEMA-designated floodplain that lies adjacent to the route.

In contrast to the Fresno Grade Separation, the right-of-way at the Jensen trench is 130 feet wide, so it is likely that the trench structure would be constructed in open cut or with minimal use of sheet pile shoring. This also means that buoyancy effects could be counteracted by incorporating a heel to the base slab without needing to increase the right-of-way.

The depth below grade never reaches the point where bracing is necessary.

3.1 Structure Importance Classification

TM 2.3.2 paragraph 2.2.1 defines all structures supporting the high-speed tracks to be primary structures because they must be reinstated after an earthquake to allow resumption of train service. The structure is also noted as Non-Standard.

This classification implies the following:

- Design life is 100 years
- Seismic design must comply with TM 2.10.4; however, the seismic design criteria for the Fresno Area indicate a PGA of less than 0.35g. In accordance with TM 2.9.10 clause 6.10.13, this means that additional earthquake pressures can be disregarded for the design of this structure.
- When applying the AASHTO LRFD code, values for the importance, ductility, and redundancy factors — h_I, h_D and h_R — have been chosen as follows:
    - Importance factor h_I = 1.05
    - Ductility factor h_D = 1.05 for strength limit states
    - Redundancy factor h_R = 1.05 for non-redundant elements, 1.0 otherwise

3.2 Key Design Features and Site Constraints

The Jensen Trench is a simple RC U-trough. These sections will be designed as rigid walls in accordance with the design criteria, which means that an “at-rest” earth pressure coefficient will be used instead of an “active” pressure coefficient. Appropriate load factors from the AASHTO LRFD code will be applied to give the design forces. The typical cross section of this configuration is shown in Figure 3.2-1.

The entire length of the HST route in the trench is greater than the 102-foot separation distance, so no additional provisions for containment of derailed UPRR trains are necessary. It is possible that shaping of the grade profile between UPRR and the trench could provide derailment containment, should this be deemed necessary.

As the trench is partially in a defined FEMA floodplain area, the trench wall height above adjacent grade has been defined as a minimum of 3 feet or 1 foot above the adjacent 100-year flood level.

The FEMA-designated floodplain extends to both sides of the HST route; on the west side the wall height is defined in the same way as the east. Additional fencing is required for fall prevention in most areas; this is not shown on the section. At the right-side boundary, access restriction fencing using independent foundations is required, which will also typically delineate the right-of-way boundary.

The right-of-way in the area of the trench has a greater width than at Fresno Grade Separation, and because the trench is relatively shallow, it is expected that the contractor will use a temporary open cutting or possibly a shallow retained cut. There is space within the right-of-way width to construct a small heel to the wall to provide resistance to buoyancy effects, should it be necessary. The relevant directive drawing DD-ST 010 limits the width of heel to a maximum of 5 feet.

The drainage design assumes that swales will be provided adjacent to the U-trough.

The section also indicates OCS equipment, which would be mounted on top of the walls using a standard portal framework.

The trench never becomes deep enough to require mounting the OCS on the sidewalls; however, if OCS supports are required where the right-of-way is constrained and adjacent to other structures, it may be prudent to provide a 10-foot-high wall to prevent a touching hazard. This situation occurs at the point where Jensen Avenue Bridge crosses over the HST.

The OCS equipment is not part of the civil engineering contract; however, knowledge of its location is required in order to finalize the design of the wall in these areas.

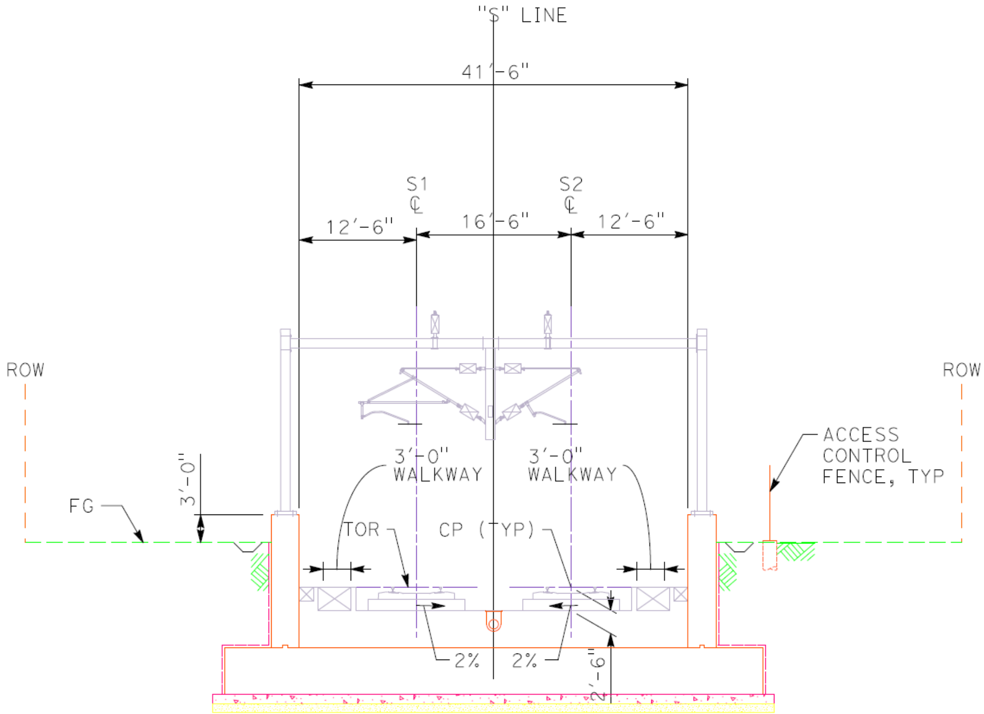


Figure 3.2-3
Typical Section of Un-braced U-Trough

3.2.1 Design Assumptions

3.2.1.1 Locked in Force

In accordance with the design criteria, the U-trough walls have been designed as rigid walls subject to at-rest earth pressures. It is assumed that temporary shoring will not be required, so no additional force would be locked in, apart from the pressures from compaction of the backfill to the structure.

3.2.1.2 Groundwater Level
Groundwater levels have been assumed to be generally below the level of the excavation. This assumption is supported by the results from the ground investigation, which generally confirmed groundwater to be at a depth of over 60 feet. In the area of the FEMA-designated floodplain, it is understood that Fresno Metropolitan Flood Control District (FMFCD) has made improvements to address the flood risk. However, the FEMA map remains unchanged and the structure design has consequently allowed for a flood water level of 1 foot above ground level.

3.2.1.3 Surcharge Pressure
To the east side, BNSF's right-of-way abuts the HST right-of-way. This will preclude use of the land for any other purpose. In areas where the route passes between S Railroad Avenue and Golden State Boulevard, surcharges are possible because the right-of-way width is greater and because the properties taken by the route may include saleable parcels of land or only partial purchases.

The BNSF tracks are generally greater than 100 feet from the trench, so the risk of additional surcharge from derailment is low.

At its current location, the UPRR adds little to the force applied to the wall. The maximum contact pressure of the Cooper E80 loading (driving wheels) is 1,882psf at the underside of ties. This pressure was applied to the wall using the Boussinesq formula. The resulting moment effect at the base of the stem was back calculated to an equivalent uniform surcharge. This procedure has demonstrated that a uniform surcharge of 420psf (equivalent to a fill depth of 3.86 feet of fill) would be adequate allowance for the Cooper E80 load and any short-term derailment surcharge.

Where adjacent land is available for potential development, a surcharge of 600psf has been applied as required by TM 2.3.2 clause 6.4.4.

3.2.1.4 Methods of Counteracting Buoyancy
The concept for the Jensen Trench follows the same logic as discussed for the Fresno Grade Separation. However, as the right-of-way width is greater and the deepest trench depth is of the order of 6 feet from grade to top of rail, it is possible to construct the heel detail as per draft directive drawing DD-ST-010 (Dated 01-17-12). Using ground investigation data available for the route, it is possible to see that the water levels are very low and there is little chance that the trench will ever be exposed to buoyancy effects from groundwater directly. The presence of hardpan at a depth of 15 to 20 feet, however, means that the trench may be exposed to local perched water tables. In a significant flood event, it is possible that water will be standing in the floodplain area long enough to generate hydrostatic pressures on the base of the trench. The design has considered this possibility, and the maximum width of heel has been used to ensure adequate resistance to uplift. Where there is insufficient resistance, this may be enhanced by thickening the base slab, thickening the walls, or backfilling with CLSM fill so that the full volume of backfill can be mobilized.

3.2.2 Key Constraints

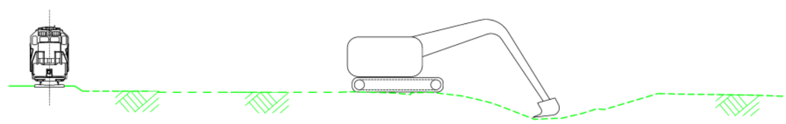
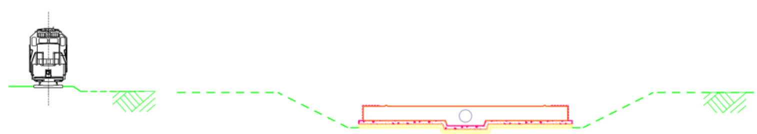
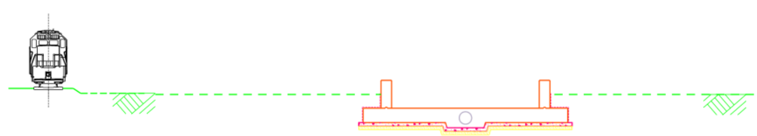
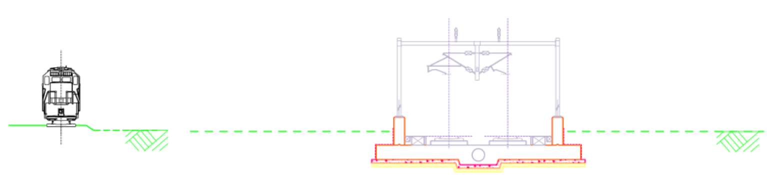
Some constraints apply to the trench as a whole, while others are design and construction constraints that may apply to only one component of the structure. The key constraints on the trench include, but are not limited to:

- The horizontal and vertical clearances from the HST track to the soffit and foundations of the existing E Jensen Avenue bridge
- The horizontal and vertical clearances from the HST track to the soffit and columns of the SR41 bridge
- The provision of a suitable length of track at constant grade to allow for 2 crossovers located within 1 mile of the station.
- Provision of features to exclude floodwater from the route (either walls or levees)

- The width of the right-of-way is generally 130 feet As the required minimum width for the track alignment and equipment is 41.5 feet, this leaves at least 88.5 feet for the following:
    - o All permanent retaining walls
    - o Temporary excavation slopes and shoring required for construction
    - o Boundary controls required to delineate the right-of-way boundary (boundary fence, intrusion protection, intrusion detection, etc.)
    - o Drainage (swales and channels)
    - o Drainage sump access
    - o Access stairs

The assumed construction stages are shown in the Table 3.2-2.

Table 3.2-2
Assumed Construction Stages

Construction Stage	**Stage Diagram**
Stage 0 Excavate to approx. 1 foot above foundation level.	
Stage 1 Trim excavation and construct base slab.	
Stage 2 Construct walls and backfill.	
Stage 3 Complete track works.	

3.3 Limits of Standard Bridge Design and Special Bridge Design

Standard bridge designs are not appropriate to this structure and the structure does not meet the criteria for a Special Bridge.

3.4 Construction Methods Assessment

3.4.1 Main Trench

The design team considered construction of the Jensen Trench in the Constructability Memo at the 15% design stage. The procurement design has undertaken outline calculations based on assumed construction sequence to confirm the adequacy of these assumptions.

3.4.2 Alternatives Considered

During the initial work at the procurement stage, the concept for the trench was reviewed and it was thought possible that an open cut solution with levees to resist the flood water may be feasible. The potential advantages of an open cut would be simpler construction and significant construction cost savings. It was agreed that the idea should be investigated further to determine whether there were any reasons not to use a cutting.

This study reported that there were many reasons for adopting a cutting, but also a number of major reasons for not adopting. These points 'for' and 'against' were discussed with the EMT and it was agreed that the trench solution should be retained.

The summary of points for and against the cutting are listed in Table 3.4-1 below.

Table 3.4-1
Comparison of Trench v Cut by Discipline

Discipline	**Cutting**	**Trench**	**Comments**
Alignment	Vertical and horizontal alignment is fixed by external constraints.	Vertical and horizontal alignment is fixed by external constraints.	Neutral
Structures	A cutting would remove the trench structure entirely. A retaining wall may still be required at Jensen Avenue. An intrusion protection wall may need to be added to protect the crest of the cutting slope.	A trench is a robust solution that satisfies the Authority's requirements but is a substantial structure to construct.	Moderately in favor of cutting option
Geotechnics	Floodwater infiltration requires significant and extensive works to prevent or accommodate inflow. Permanent and enhanced pumping capacity is required.	Novel design methodologies not required. The trench can be designed to accommodate hydrostatic forces using a number of simple and commonly used methods.	Strongly in favor of trench option
Drainage	Significantly increased storage and pumping requirement over baseline	Baseline storage and pumping requirement	Strongly in favor of trench option
Utilities	Utilities can be accommodated by using pumps, siphons, or diversions. Design variance required (utility within 8 feet of TOR).	Utilities can be accommodated by using pumps, siphons, or diversions. Design variance required (utility within 8 feet of TOR).	Neutral
Right-of-Way	Increased right-of-way required	Baseline right-of-way requirement	Moderately in favor of trench option
Operations	Equal protection provision compared to baseline. Increased risk from external derailment impacting HST operations.	Baseline collision protection and risk to operations	Moderately in favor of trench option
Cost	Significantly reduced base cost compared to baseline. Additional costs from drainage infiltration works, collision protection wall, levee construction, additional detention basin capacity. Additional lifetime costs from cost of pumping and maintenance.	Baseline cost	Strongly in favor of cutting option

3.5 Temporary Construction Loadings Considered

During the construction of the Trench, a number of temporary construction loads will be present for short or long periods. As the trench is envisaged as being constructed in open cut, it is unlikely that any of these loads will affect the design of the structure. Should the right-of-way restrict the full excavation in open cut e.g. at Jensen Avenue Bridge it may be necessary to use localized sections of shoring wall. In such cases, the shoring should be designed to allow for the following additional loads where appropriate:

- A surcharge pressure of 600psf applied to areas where construction activity may use land adjacent to the shoring.

3.6 Temporary Construction Easements

Temporary construction easements are required for the construction of the following:

- Connections to the trench drainage sump
- Utilities diversions

The drainage sump is located close to the E Jensen Avenue Bridge and will be connected to the local drainage system via a new detention basin. The basin will be constructed adjacent to the southern end of the trench.

3.7 Traffic or Pedestrian Diversion and Control

The construction of the trench requires the permanent closure of S Railroad Avenue (part), E Florence Avenue, S Sarah Street, S Belgravia Avenue, S East Avenue and S Orange Avenue.

Traffic management will be necessary to accomplish these changes. The contractor will be required to prepare and implement a Construction Transportation Plan in close coordination with the City of Fresno as described in Section 3.2.6, Project Design Features, of the Merced to Fresno Final EIR/EIS.

For the construction of the U-trough, there will be a need for construction entry and egress points that connect to the road system. It is expected that the majority of excavated material from the U-trough will need to be taken offsite via these egress points. It will be necessary to agree upon the amount, frequency, and operating hours for these entry/egress points with the City of Fresno. It is possible that some of the excavated material could be suitable for re-use as embankment fill for the Fresno Viaduct approaches.

3.8 Drainage Concept

The track drainage within the trench will be carried in a single longitudinal pipe as per the draft directive drawing No (DD-ST-010 dated 01/17/12). Subsequent discussions with PMT/EMT have permitted the drain to be cast into the structural base slab with an additional haunch where the pipe diameter exceeds the slab thickness. Grades less than 0.25% will require a separate drainage system in accordance with TM 2.1.2. As the alignment grade is less that 0.25%, the trench has therefore been detailed to allow the drains to be placed adjacent to the structure to a greater gradient.

A typical section of this arrangement is shown below:

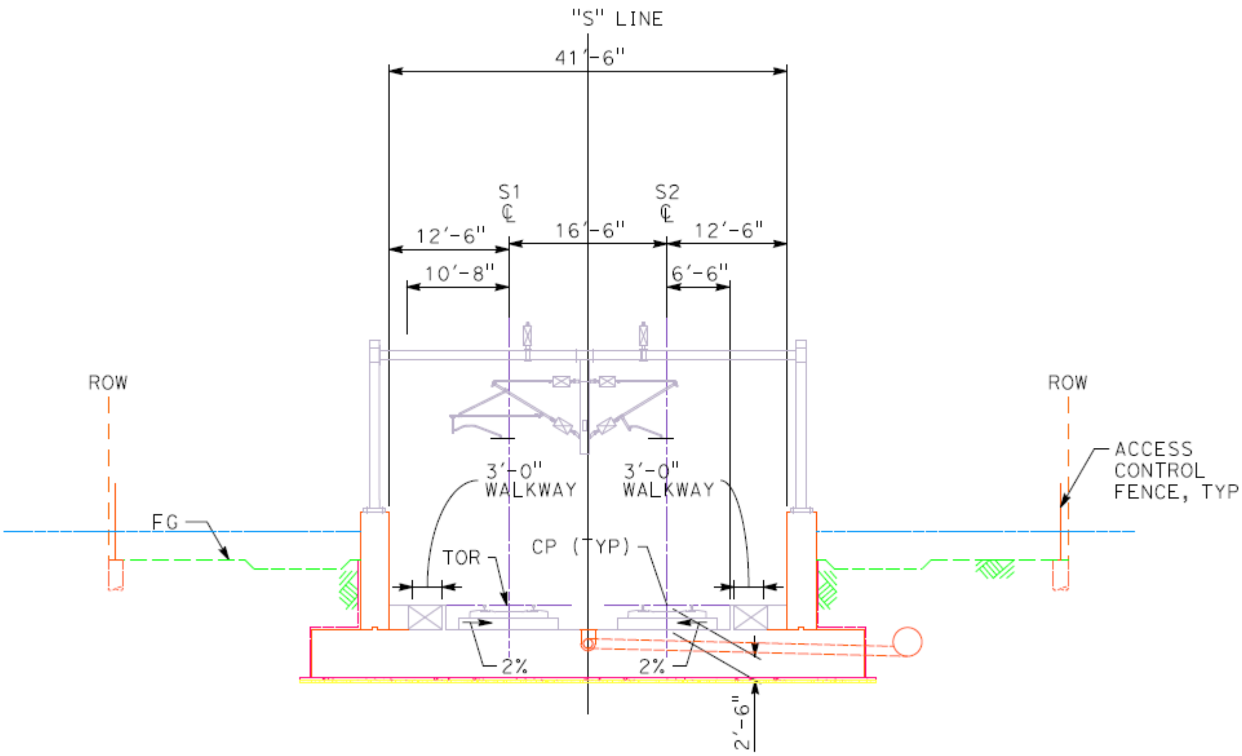


Figure 3.8-1
Typical Section of Jensen Trench

At the low point on the approach to the sump, the drainage pipe reaches a diameter of 3' – 6".

3.9 Emergency Egress and Escape Provision

Although not strictly an elevated or underground facility, the team has agreed that it is appropriate to apply the requirements of NFPA130 for emergency escape/egress to the U-trough. This means that escape stairwells are to be provided at maximum 2,500-foot intervals through the trench. Stairwells are provided as indicated in Table 3.9-1.

Table 3.9-1
Stairwell Provisions

STA	Locale	Egress features
111095+00	The south side of South East Avenue	Stairwell is located within the current property boundary of a Propane store.
11120+00	Located adjacent to the drainage sump. Vehicular access from Golden State Boulevard.	Emergency services access to the location of the stairwell will need to be agreed with the owner of the facility. There is space for provision of a turning area for vehicles.

Each stairwell is 5 feet wide by 25 feet long to allow for the later installation of a single-flight staircase.

The staircase is assumed to be 44 inches minimum width. No landings are thought to be required. As the trench is located in a floodplain, the stairs should rise to the top of wall level before descending back to grade.

3.10 Inspection, Service, and Maintenance Access

The trench structure itself will be a simple massive RC structure with a limited number of movement joints at intervals. There will be no specific provision for inspection or maintenance access other than the general maintenance access to the route.

The drainage sump will require maintenance access at the surface and access for the installation and removal of pumps. Pedestrian access will also be provided by construction of an access door from the walkway within the trench. Providing this door increases the risk that it may be dislodged by the passage of a train, so it is proposed that this door and the doors associated with the emergency escape stairs should be sliding doors. These may be fitted during a later contract.

Access for pump maintenance and replacement will require a permanent easement and is likely to use the same access point as the emergency egress stairs near to Jensen Avenue.

Movement joints in the walls will be required to limit the effects of temperature and ground movement. These joints are intended to be no more complex than simple cast-in waterstop details.

3.11 Utilities Affected and Disposition

A number of existing utilities cross the route of the trench or are within the proposed right-of-way. Where these can be diverted, the proposed diversion route has been identified on the utilities and structures layout drawings. It has been a principle of this work to divert utilities around or away from the HST route where possible. Where there are specific crossing points that cannot be accommodated in this way, the utility has been diverted under the trench.

Examples of where the trench design may be affected are as follows:

- **86-inch Storm Drain at Church Ave (STA 11086+00)**
  This line is part of the FMFCD network of storm drains. It may be possible to divert the drain around the trench, but the length of diversion is long because of the restricted falls available. It has been assumed, that this utility will be reconstructed. The pipe will be sleeved under the Trench to permit future removal and replacement.
- **30-inch Sewer Line at Church Ave (STA 11086+00)**
  As with the storm drain, there appears to be no reasonable alternative route. It is proposed, that the pipe will be reconstructed on line to pass under the trench. The pipe will be sleeved under the Trench to permit future removal and replacement.
- **48-inch Sewer line at Jensen Avenue (STA 11122+00)**
  This sewer line cannot avoid crossing the route of the HST, there appears to be no reasonable alternative route. It is proposed, that the pipe will be split into two 36" pipes and diverted parallel to Jensen Ave. The pipes will be sleeved under the Trench to permit future removal and replacement.

3.12 Hydrological Issues

These issues are discussed in detail in the Floodplain Impact Assessment Report.

The main impact of the trench design is to ensure that the trench wall is higher than the 100-year flood level in the FEMA designated floodplain. In this area, the 100-year flood level is approximately 1 foot above ground level. Protection against flooding will be provided by the trench wall, which projects above grade level by a minimum of 3 feet.

3.13 Noise Mitigation and Acoustic Treatment

The Merced to Fresno Final EIR/EIS provides information on operational noise mitigation requirements that have been adopted by the Authority. Implementation of operational noise mitigation is not part of the scope of this design build contract. However, project facilities that will be completed under this contract must be designed to accommodate future noise mitigation elements.

3.14 Details of the Geotechnical Parameters Used for Design

The geotechnical parameters are described in the Geotechnical Design Memorandum attached at Appendix A.

4.0 The Fresno Viaduct

The Fresno Viaduct is composed of 49 spans, most of which are of standard 100- to 120-foot-span post-tensioned concrete box girders. However, at the crossings over Golden State Blvd, S Cedar Ave. and SR99, the standard spans are unable to provide a solution due to the large skew angle to the obstacle crossed. In these locations, steel truss spans have been detailed.

The span lengths for the Golden State Blvd and S Cedar Ave crossings are 315 feet, 355 feet respectively. The SR99 crossing has been detailed as a two-span truss structure which is continuous over an intermediate bent; both spans are 245 feet.

The foundations of the viaduct are formed using drilled shafts, topped by 10 foot thick RC pile caps. The size of the pile groups vary from four piles to seven piles, with the larger pile groups detailed for the footings of the truss structures.

Columns supporting standard spans are made up of single, circular RC sections. Clear heights vary along the length of the viaduct and range from a minimum of 10’-1” to a maximum of 38’-11”. Section sizes of the columns are specified as 8 feet diameter for clear heights less than 29 feet and 10 feet diameter for clear heights in excess of 29 feet. The tops of each column have a column cap that provides seating for the bearings.

The Golden State truss structure is supported at its northern end by two RC pilasters, acting compositely with an RC wall. The remaining supports for the truss superstructures with the exception of the intermediate support on the SR99 crossing, are formed by RC bents. The bents are made up of two 10 feet diameter columns and RC capping beam.

The SR99 intermediate support is formed by a single RC column, similar to the standard span supports with the exception that a 14 feet section diameter has been specified. The footing in this case is also unique in that the pile cap is situated at a skew to the capping beam, in order to match the SR99 carriageway alignment.

The southernmost abutment consists of a single RC pilaster and an RC wall, which forms the interface between the viaduct and MSE embankment.

The right-of-way width for the viaduct is 60 feet wide.

4.1 Structure Importance Classification

The design criteria define all structures supporting the high-speed tracks to be primary structures because their reinstatement is necessary to permit resumption of train service after an earthquake.

The majority of the length of the viaduct requires columns that are less than 30 feet high. These spans are in accordance with the standard. However, during this design, the range of column heights has been extended to 40 feet, so all spans are standard except for those supporting the trusses.

These classifications imply the following:

- Design life is 100 years.
- Seismic design must comply with TM 2.10.4; however, the seismic design criteria for the Fresno area indicate a PGA of less than 0.35g. In accordance with TM 2.9.10 clause 6.10.13, this means that additional earthquake pressures can be disregarded for the design of this structure.
- When applying the AASHTO LRFD code, values for the importance, ductility, and redundancy factors — h_I, h_D, and h_R — have been chosen as follows:

Importance factor h_I = 1.05

Ductility factor h_D = 1.05 for strength limit states (1.0 for conventional designs)

Redundancy factor h_R = 1.05 for non-redundant elements, 1.0 otherwise

4.2 Key Design Features and Site Constraints

At the north end of the viaduct the HST is supported on MSE wall as indicated in Figure 4.2.-1 The north abutment terminates the retained section with an RC wall that is composite with two pilasters that support the truss of span 1 (Golden State Boulevard).

The dimensions of the truss were modified to accommodate the horizontal radius of the HST alignment, resulting in the widening of the sections. The Golden State Blvd and S Cedar Ave/SR99 structure section widths have therefore been specified as 41.5 feet and 42.25 feet respectively, taken as the distance between the centerlines of the bottom chords.

The Golden State Blvd and S Cedar structures are both formed by steel trusses with curved top chords. The depths of the truss sections, taken as the distance between the centerlines of the chords, vary at both locations between 35 feet adjacent to the supports and up to 50 feet at midspan.

The SR99 crossing is a two-span truss which is continuous at an intermediate support, situated in the SR99 central reserve. The truss sections maintain a uniform depth, measured as above, of 35 feet along the full length.

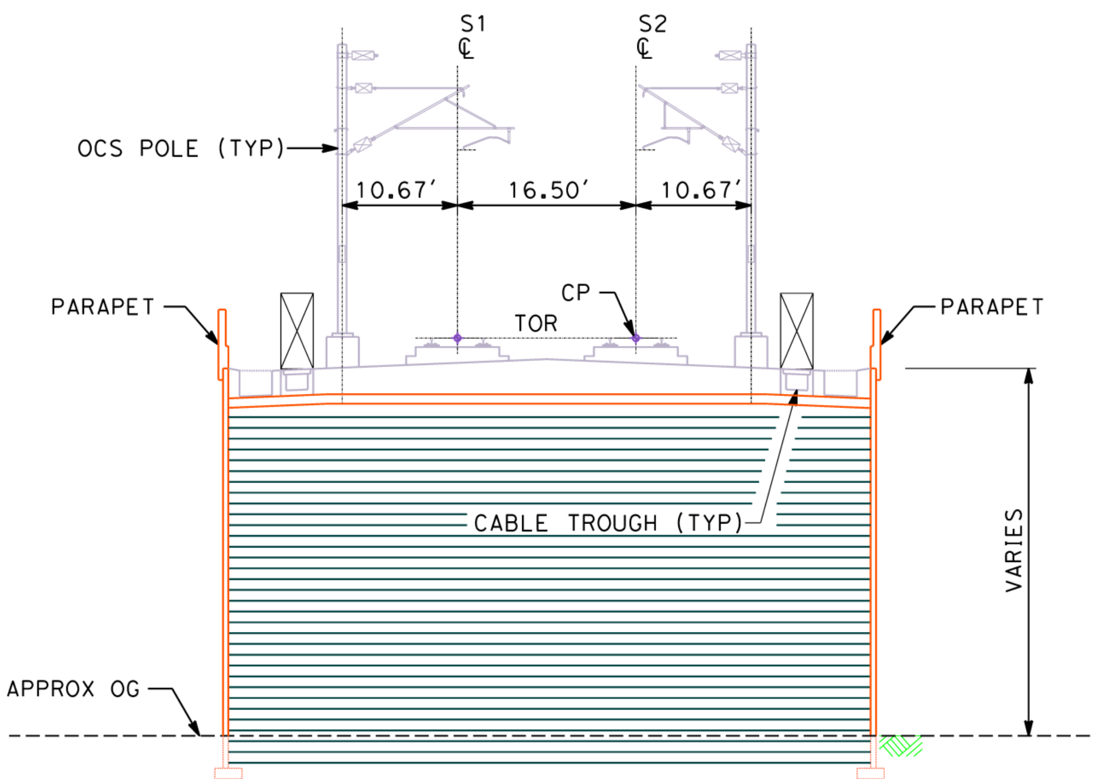


Figure 4.2-1
Typical Section of MSE Embankment

The truss has been configured so that the deck slab acts compositely with the lower truss chord members and is composite with the cross girders.

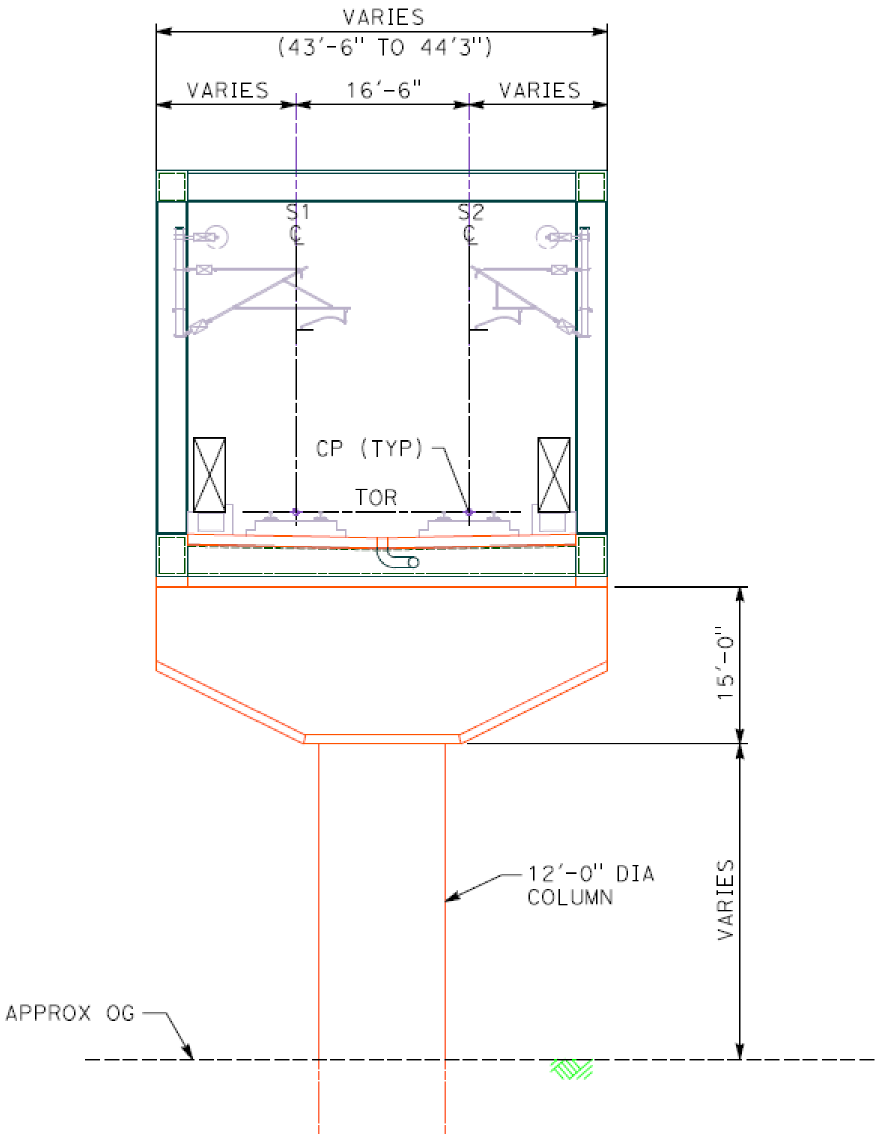


Figure 4.2-2
Typical Section of Truss Span

4.2.1 Design Assumptions

For design, the requirements of the design criteria dictate that the truss spans at S Cedar and SR 99 cannot be analyzed in isolation. Therefore, a single model of the structure was constructed that combines the Truss spans with 10 standard spans to either side. The model also includes elements to represent each track with the rails modeled as single elements supported by nonlinear springs to represent the track clips.

The purpose of the model is to calculate structure forces and displacement, to model the effect of track structure interaction on the movement joints and on rail stresses, and to provide information to confirm the capacity of the foundations. The properties of the ground are incorporated in the structure model with the use of equivalent springs which reflect the soil parameters, given in Appendix A.

Seismic analyses consisted of non-linear time-history analysis for OBE events, to assess structural adequacy and track-structure interaction limits; and non-linear time-history analysis for MCE events, to assess the displacement demands. Pushover analyses were performed locally on the columns to determine the performance of plastic hinges and evaluate ductility. This procedure has been described in further detail in the Seismic Analysis Plan (see Appendix B).

4.3 Summary of Analysis and Results

For the purposes of the analysis, the structure was separated into 3 sections: the road crossing of Golden State Blvd; the road crossing of S Cedar Ave. and SR99; and the southern abutment. The RC box girder spans are classified as standard structures and so do not fall within the scope of the preliminary design, although they have been modeled where necessary in accordance with the Seismic Design Criteria.

All sections have been checked for resonance effects, rail serviceability and track-structure interaction limits, and force demands. In all cases the structure has been found to be satisfactory. For conciseness, only the frequency results are presented in this report. Refer to the Fresno Viaduct Calculations report for the complete analysis and results.

Based upon the calculations thus far it appears that the designs are in full compliance with the TMs and are capable of being developed into a fully compliant design solution. Refer to the calculation package for the complete analysis and results.

4.3.1 Modeling

Both SAP2000 V14 (SAP) and CSiBridgeV1520 (CSiBridge) modeling programs were used for the analysis of the Fresno Viaduct. Several models of each section were required in order to represent the different conditions of the structure at different loading cases and for different design checks, in accordance with TM 2.10.4 and 2.10.10.

The structural columns, truss members, rails and RC girders were represented by stick elements. Piles were represented by non-linear springs, using equivalent stiffness values to correctly model the soil structure interaction based on soil parameters in Appendix A. The pile cap and pile group effects were modeled using rigid links connecting the top of the piles to the column elements. All spans were connected to the bent cap elements with linear bearing springs, with the bridge articulation represented by either pinned or rolling spring properties. In the unique case of the transverse frequency analysis, rigid restraints were added in place of the bearings, as only the flexibility of the superstructure needed to be considered.

Foundation arrangements for the standard spans were provided by the PMT and have been used accordingly in the structural models. These foundations have been checked using LPILE and Pilset and found to have adequate capacity.

4.3.2 Frequency Results

The vertical, torsional and transverse frequencies of the structure were evaluated to ensure that they meet the required train serviceability criteria, as defined by TM 2.10.10. In the case of the vertical and torsional frequencies, two conditions were assessed: the first with upper bound mass and lower bound stiffness (Condition 1), the second with lower bound mass and upper bound stiffness (Condition 2). Condition 1 was also adopted for the transverse frequency analysis, as this generated the smallest and therefore most onerous frequency results in this case.

In all three sections of the Fresno Viaduct, the natural frequencies were found to be within the defined limits. See tables 4.3-1 through 4.3-4 for the results.

Table 4.3-1
Golden State Blvd Frequency Check Results

	Vertical Frequency (Hz)	Torsional Frequency (Hz)	Transverse Frequency (Hz)
Lower Limit	1.58	2.56 (C1), 2.70 (C2)	1.2
Upper Limit	3.12	N/A	N/A
Condition 1	2.14	2.58	2.03
Condition 2	2.25	2.74	N/A

Table 4.3-2
S Cedar Ave Frequency Check Results

	Vertical Frequency (Hz)	Torsional Frequency (Hz)	Transverse Frequency (Hz)
Lower Limit	1.49	2.28 (C1), 2.35 (C2)	1.2
Upper Limit	2.88	N/A	N/A
Condition 1	1.90	2.40	1.68
Condition 2	1.96	2.47	N/A

Table 4.3-3
Southern Abutment Frequency Check Results

	Vertical Frequency (Hz)	Torsional Frequency (Hz)	Transverse Frequency (Hz)
Lower Limit	1.65	3.03 (C1), 3.19 (C2)	1.2
Upper Limit	3.28	N/A	N/A
Condition 1	2.52	3.27	2.47
Condition 2	2.65	3.56	N/A

Table 4.3-4
Southern Abutment Frequency Check Results

	Vertical Frequency (Hz)	Torsional Frequency (Hz)	Transverse Frequency (Hz)
Lower Limit	2.80	4.63 (C1), 4.80 (C2)	1.2
Upper Limit	6.42	N/A	N/A
Condition 1	3.86	8.27	16.25
Condition 2	4.00	8.82	N/A

4.4 Limits of Standard Bridge Design and Special Bridge Design

It is assumed that the standard bridge design is suitable for use on spans 2 to 32 and from spans 36 to 49. Span 1 and spans 33 to 35 are considered in this design.

4.5 Construction Methods Assessment

The three locations where truss structures are planned each have specific features that suggest a particular method of erection is most likely to be used by contractors. This does not rule out other

methods of construction. It is likely that contractors will prefer to use methods that they have used successfully in the past. The assessment described here represents a subset of methods that could be used.

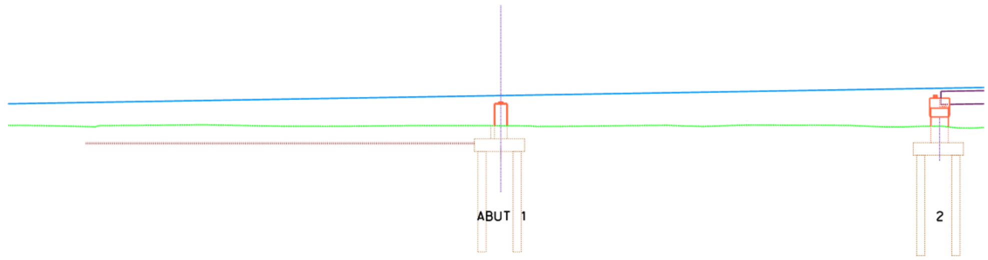
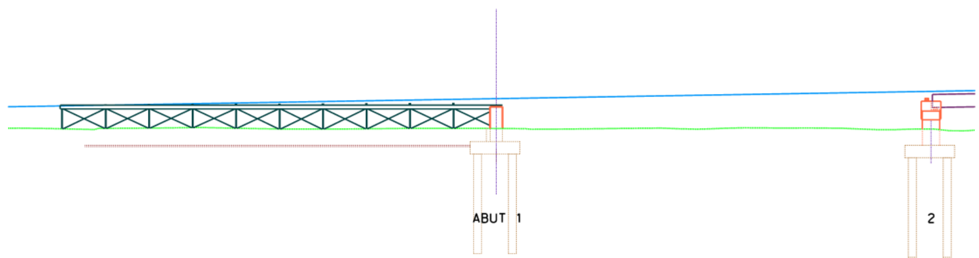
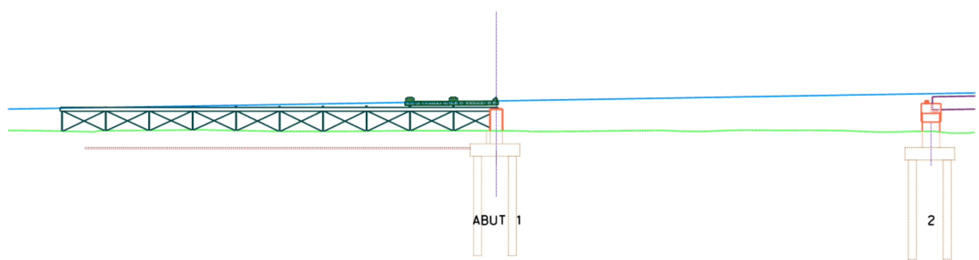
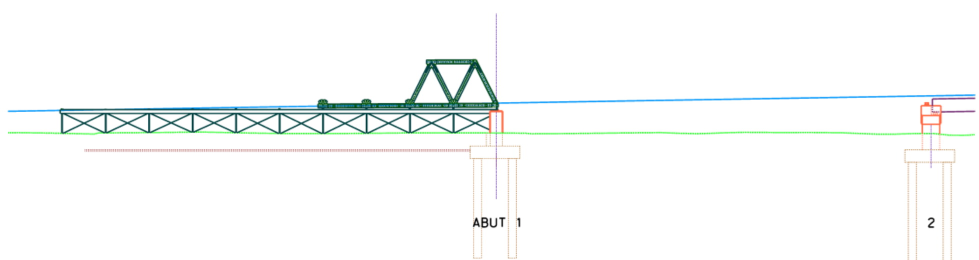
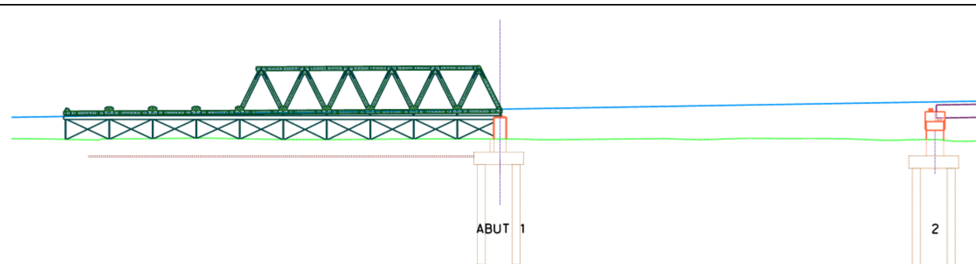
4.5.1 Golden State Boulevard

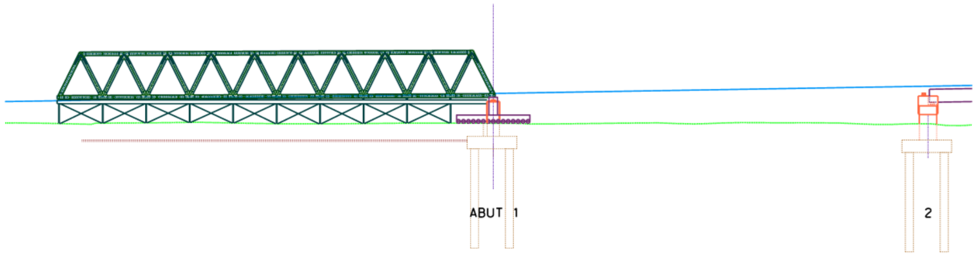
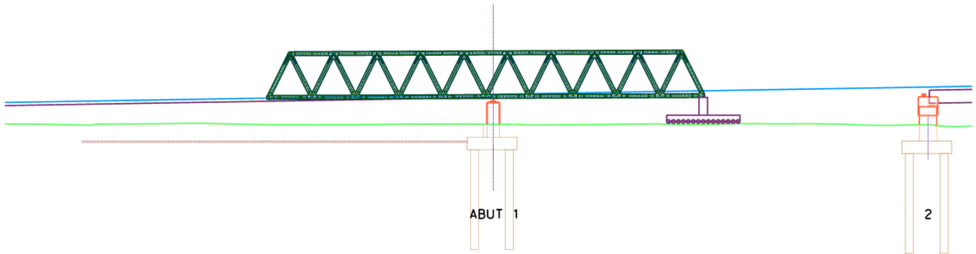
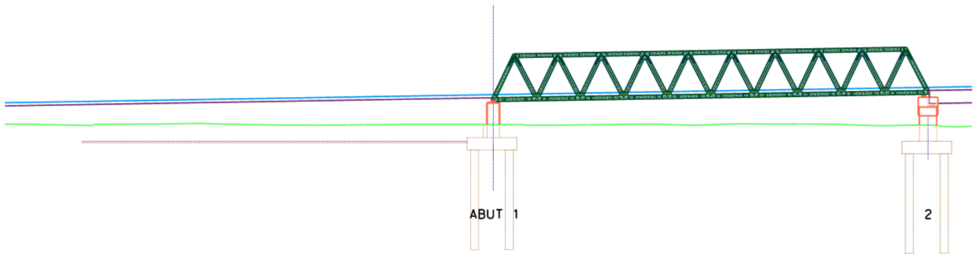
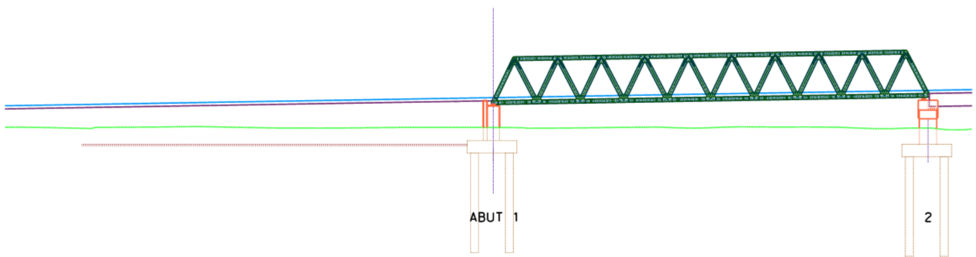
It is assumed that the area of land available to the north of Golden State Boulevard will be used for lay-down and assembly of structure components for the truss. The truss itself will be assembled on the line of the HST prior to the construction of the approach embankment and close to its final height supported on temporary trestles. Once fully assembled the truss will be lifted using a heavy lift vehicle and transported across Golden State Boulevard to be lowered onto its bearings on the far side.

The basic construction sequence is shown in Table 4.5-1 and is as follows:

- Construct temporary trestles as supports (Stage 1 and 2)
- Erect steelwork superstructure (Stage 3 to 5)
- Clear obstructions for heavy lift vehicle (Stage 6)
- Launch truss to contact with heavy lift vehicle
- Heavy lift vehicle transports the end of the bridge across GSB to prepared column bent No. 2 in Southern shoulder of GSB (Stages 7 & 8)
- Dismantle trestles and construct abutment wall and embankment (Stage 9)

Table 4.5-1
Golden State Boulevard - Construction Sequence

Stage 1	
Stage 2	
Stage 3	
Stage 4	
Stage 5	

Stage 6	
Stage 7	
Stage 8	
Stage 9	

4.5.1.1 South Cedar Avenue

The layout of the SR 99 junction and the HST suggests that the land to the east of S Cedar Avenue and within the junction could be used to erect the truss steelwork on trestles close to its final height. In parallel, a temporary slide track would be constructed to span S Cedar Avenue while still permitting traffic flow. On completion, the truss would be rotated or slid across S Cedar Avenue to land on its final bearing supports. This operation would be completed during a full closure of S Cedar Avenue.

Two alternatives for the slide process are indicated in Figures 4.5-1 and 4.5-2. The construction sequence is as follows:

Construct temporary trestles as supports

- Erect steelwork superstructure
- Erect radial slide track across S Cedar Avenue
- Launch/rotate structure along slide track to land at pier Bent Nos. 33 and 34
- Dismantle slide track and trestles

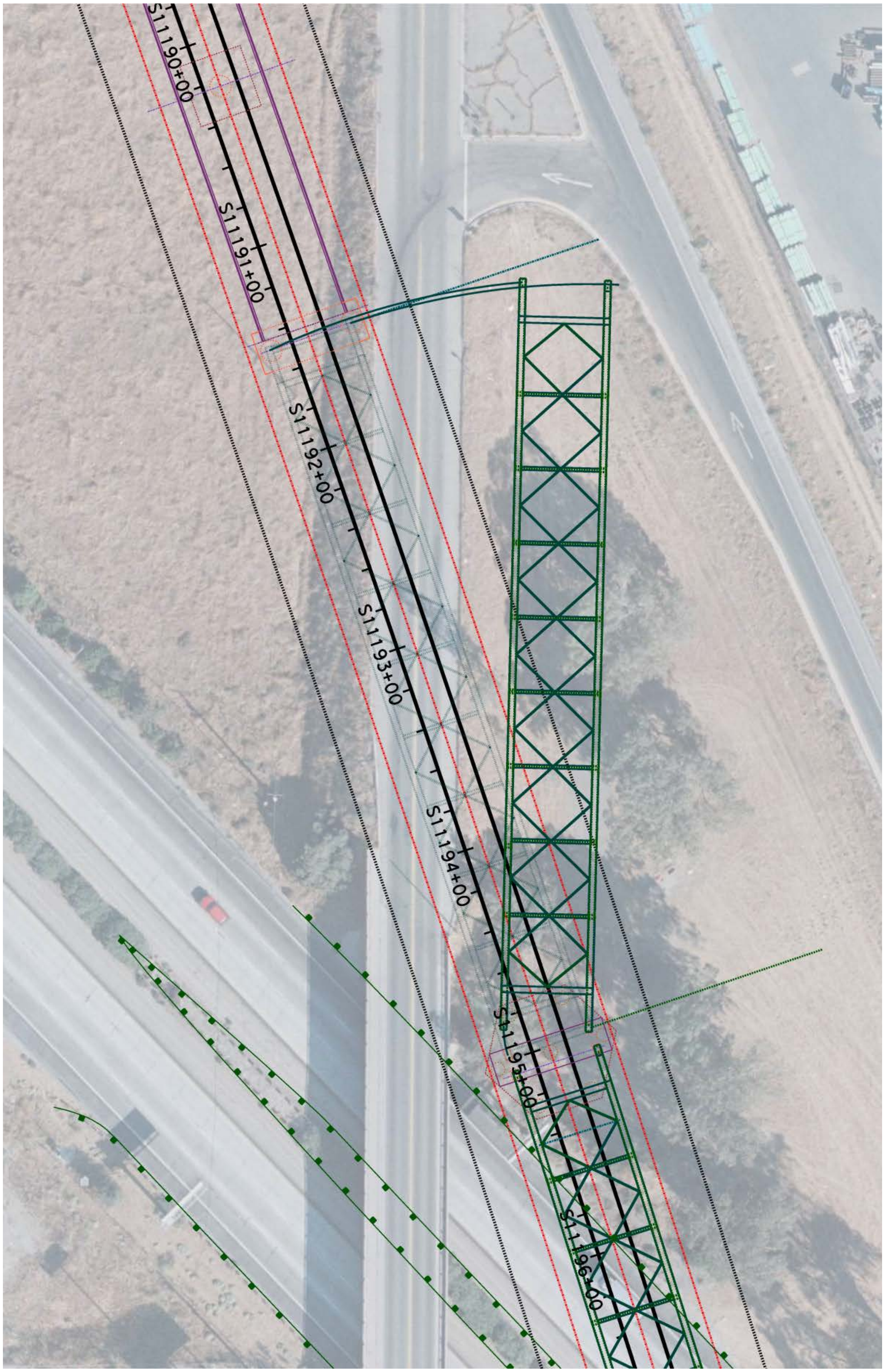


Figure 4.5-1 Erect Truss and Rotate into Position

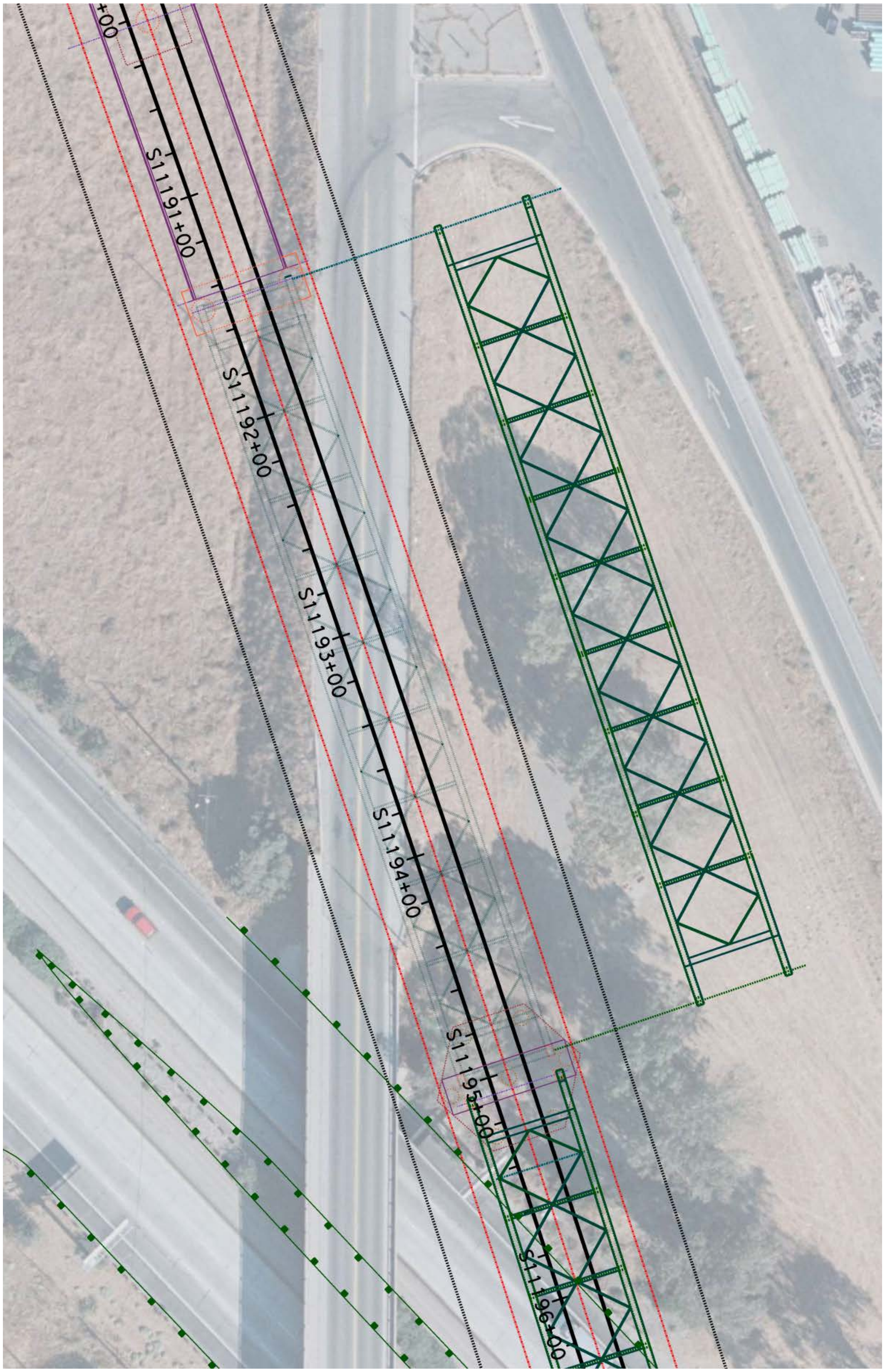


Figure 4.5-2 Erect Truss and Slide into position

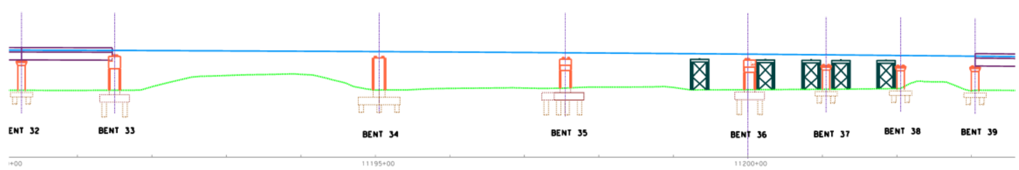
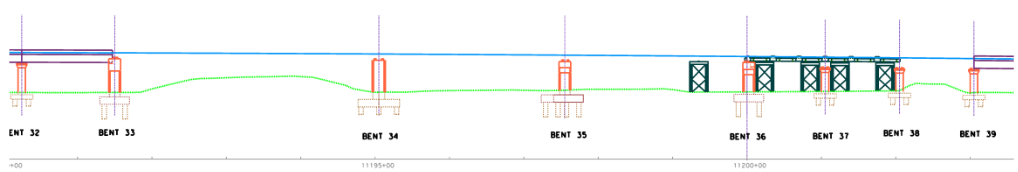
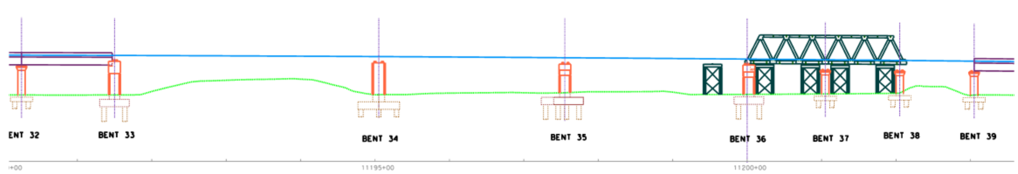
4.5.1.2 SR 99

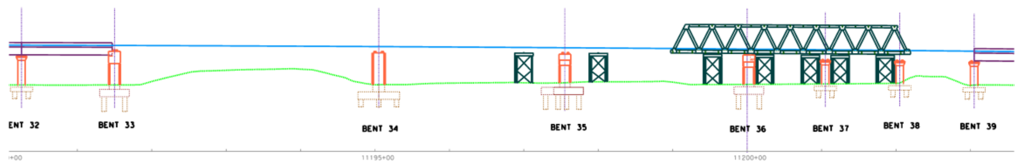
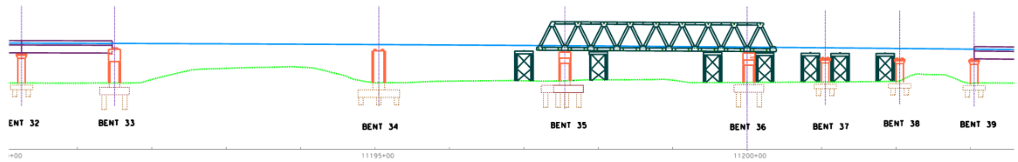
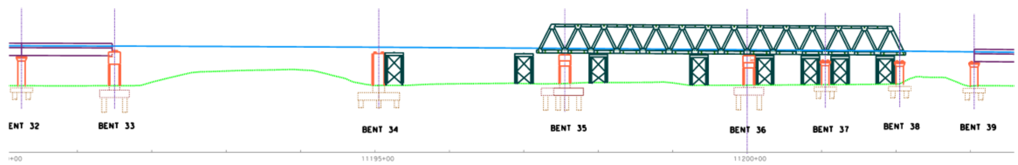
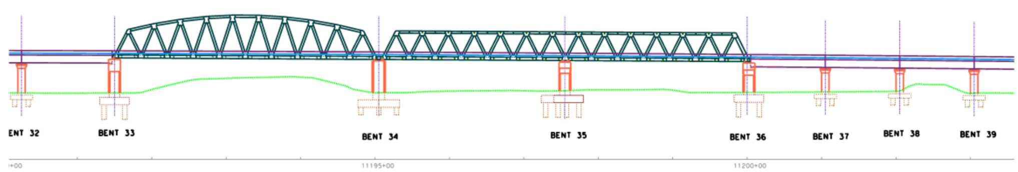
The layout of the SR 99 junction to the south of the SR 99 permits space for construction between the main route and the southbound on ramp. This space is insufficient for the assembly of the full length of the truss, so incremental launching has been assumed. Discussions with Caltrans have indicated that movement of the structure will not be permitted under live traffic flow. Temporary closures will therefore be required during the launching stages of the program.

The basic construction sequence described below and shown in Table 4.5-2 is as follows:

- Construct temporary trestles as supports between on ramp and main route and also in shoulders and median of SR 99 (Stage 1)
- Erect steelwork superstructure for initial section of truss to the south of SR 99 (Stages 2 and 3)
- Launch steelwork onto temporary trestles (during a temporary closure of all or part of the SR 99) (Stage 4)
- Erect next stage of truss steelwork in southern cutting of SR99 (Stages 5 and 6)
- Repeat until structure complete
- Dismantle trestles (Stage 7)

Table 4.5-2
SR99 Construction Sequence

Stage 1	
Stage 2	
Stage 3	

Stage 4	
Stage 5	
Stage 6	
Stage 7	

This method of construction has been used on other projects with similar constraints as can be seen in the following photos from Kuala Lumpur.

Figure 4.5-3 Erection of Truss Bridge by Incremental Launching

Figure 4.5-4 Finished Structure

4.6 Temporary Construction Loadings Considered

No specific loadings have been considered for the temporary stages described. It is likely that additional temporary bracing may be required to prevent excessive distortion.

For incremental launching, the contractor will be required to provide calculations that support the proposed launching sequence and methodology.

4.7 Temporary Construction Easements

A general temporary construction easement of 100 feet width has been indicated for the full length of the viaduct. This should be sufficient for the foreseeable requirements for construction of such a structure.

Additional temporary construction easements may be required for the construction of the following:

- Storm drain diversions
- Colony Canal diversion
- Foundation construction adjacent to Caltrans facilities (SR 99 median and shoulders)
- Connections to the trench drainage sump
- Emergency egress stairwells and emergency access roads

4.8 Traffic or Pedestrian Diversion and Control

Closures of the SR 99 will not be permitted without a viable diversionary route. Caltrans will require the identified route to be clearly signposted for users.

4.9 Drainage Concept

The track drainage for the Fresno Viaduct will be carried from deck level through to a permanent drainpipe fitted within the void of the concrete deck girders. This pipe will be connected to downpipes cast into the columns. The downpipes will outfall near ground level to the surface drainage system. For the steel truss spans, provision will be made for collecting water at track level. This will be conveyed to the ends of the structure via a longitudinal carrier pipe that will be sleeved through the transverse girders of the trusses. At the ends of the truss structures the carrier pipe will discharge to the nearest available downpipe as per the standard spans.

4.10 Emergency Egress and Escape Provision

Provision for emergency escape will be made in accordance with NFPA130. This means that escape stairs are to be provided at maximum 2,500-foot intervals along the viaduct, although the local fire Marshall has proposed that 3,000-foot intervals are acceptable. In assessing locations, the retained embankment has been considered as part of the structure.

Table 4.10-1
Escape Stair Locations

STA	Locale	Egress features
11152+00	Adjacent to the start of the Span 1	Staircase constructed as part of the MSE retaining wall adjacent to Abutment No 1
11177+00	Adjacent to the entrance to the Valley Wide Beverage facility	Staircase detail in accordance with the directive drawing.
11207+00	Adjacent to existing facility to be demolished.	Staircase detail in accordance with the directive drawing. Vehicular access via Muscat Avenue.

4.11 Inspection, Service, and Maintenance Access

The standard viaduct will be a simple concrete section that is inspectable from both inside and outside.

The steel truss spans are envisioned to be be constructed using hollow fabricated steel sections for the chords and I-sections for the diagonals. These are unlikely to be inspectable and should be treated internally with corrosion inhibitors or sacrificial thickness of steel. Externally, the truss structures will be painted and inspectable with the use of hydraulic access platforms.

4.12 Utilities Affected and Disposition

The major utilities that cross the route of the viaduct are as follows:

- **Colony Canal at Golden State Boulevard (STA 11156+00)**
  This canal has been culverted, probably at the time of construction of Golden State Boulevard. It runs in twin pipes along the line of the southern shoulder of Golden State Boulevard before turning southward. The southward turn clashes with the location of the foundation for Bent No. 2 of Fresno Viaduct, so it is proposed to divert the canal along the boundary line of the right-of-way to remove the foundation clash.
- **Central Canal adjacent to BNSF spur (STA 11167+00)**
  The Colony Canal is culverted beneath a BNSF spur line and on exit clashes directly with the location of Bent No 12. Rather than divert the canal or move the column, it is proposed that the channel of the canal could be locally increased in width to allow the water to flow around the column. The column pile cap will be sufficiently large to prevent any flow scour issues at the column. This will require FID agreement.
- Hydrological Issues

Hydrological issues are discussed in detail in the Floodplain Impact Assessment Report.

4.13 Noise Mitigation and Acoustic Treatment

Noise mitigation will be adopted through the environmental review process for the Fresno-Bakersfield segment. Implementation of operational noise mitigation is not part of the scope of this design build contract. However, project facilities that will be completed under this contract must be designed to accommodate future noise mitigation elements.

4.14 Compliance with Systemwide Bridge Aesthetics Features

TM 200.06 provides guidance on non-station structures. The scheme detailed on the drawings and analyzed represents the functional baseline case.

4.15 Details of the Geotechnical Parameters Used for Design

The geotechnical parameters are described in the Geotechnical Design Memorandum attached at Appendix A.

5.0 Additional Structures

Structures that span over the HST, are in close proximity to the HST, or support other non-HST facilities are listed below:

- W McKinley Avenue
- W Olive Street
- E Belmont Avenue
- Tuolumne Street
- Fresno Street Overpass
- G Street/Fresno Street Underpass
- Tulare Street Crossing (discontinued)
- Tulare Street Overpass
- Ventura Street Crossing (discontinued)
- HST/Ventura Street Underpass
- UPRR/Ventura Street Underpass
- G Street/Ventura Street Underpass
- E Church Avenue Crossing
- E Central Avenue Crossing
- E American Avenue Crossing
- Stanislaus Street Crossing
- Stanislaus Street Pedestrian Crossing

The roadway structures that span over the HST do not directly support the tracks, but they do have an impact on the operation of the HST. For this reason, they are subject to higher seismic requirements than typical structures and are therefore classified as nonstandard, primary structures.

Refer to the Package 1 Roadway and Pedestrian Structures Reports for further details.

5.1 W McKinley Avenue Crossing

The W McKinley Avenue crossing is a skewed, three-span road bridge that spans over the HST alignment, UPRR line, and N Weber Avenue. The bridge has a total length of 419 feet 9 inches, with spans ranging between 104 feet 10 inches to 164 feet 11 inches. The superstructure is formed of precast, prestressed RC girders and RC deck slab. The substructure is composed of RC abutments and two three-column RC bents.

5.2 W Olive Street Crossing

The W Olive Street crossing is a skewed, three-span road bridge that spans overs the HST alignment, UPRR line, and N Weber Avenue. The bridge has a total length of 427 feet, with spans ranging between 136 feet 4 inches to 153 feet 8 inches. The superstructure is formed of precast, prestressed RC girders and RC deck slab. The substructure is composed of RC abutments and two three-column RC bents.

5.3 E Belmont Avenue Crossing

The E Belmont Avenue crossing is a skewed, five-span road bridge that spans over the HST alignment, UPRR line, and N Weber Avenue/H Street. The bridge has a total length of 653 feet 10 inches, with spans ranging between 115 feet to 139 feet 3 inches. The superstructure is formed of precast, prestressed RC girders and RC deck slab. The substructure is composed of RC abutments and four 3-column RC bents.

5.4 Fresno Street Overpass

The Fresno Street Overpass is formed by a two-cell RC box culvert, with the central wall situated in the central reserve of Fresno Street. The roof slab is monolithic with both the side walls and central wall, which are skewed to match the alignment of Fresno Street. The total bridge length is 84’-1”.

5.5 G Street / Fresno Street Underpass

The G Street/Fresno Street underpass is a two-span road bridge of deck-type construction. The superstructure is composed of a precast, prestressed RC slab, which is supported by two RC abutments and a three-column RC bent situated in the central reserve of Fresno Street. The substructure is skewed by 2.5° to match the alignment of Fresno Street.

5.6 Tulare Street Crossing (discontinued)

The Tulare Street crossing is a six-span road bridge that spans over the HST alignment and UPRR line. The bridge has a total length of 780 feet, with spans ranging between 100 and 160 feet. The superstructure is formed of precast, prestressed RC girders and RC deck slab. The substructure is composed of RC abutments and five two-column RC bents.

5.7 Tulare Street Overpass

The Tulare Street Overpass is a single-span rail bridge that supports the UPRR line and has a total length of 59 feet. The superstructure is formed of a series of steel I-girders with an additional top steel plate to form the deck. The abutments are composed of contiguous bored piles, which form a rigid wall-to-seat the superstructure.

5.8 Ventura Street Crossing (discontinued)

The Ventura Street crossing is a six-span road bridge that spans over the HST alignment, UPRR line, and H Street. The bridge has a total length of 800 feet, with spans ranging between 120 feet and 157 feet 10 inches. The superstructure is formed of precast, prestressed RC girders and RC deck slab. The substructure is composed of RC abutments and five three-column RC bents.

5.9 HST and UPRR Ventura Street Underpasses

The HST/Ventura Street Underpass is a two-span deck-type bridge that supports the HST line and spans over Ventura Street. The superstructure is composed of a series of precast, prestressed RC box girders. The substructure is made up of two RC abutments and an intermediate RC wall, situated in the central reserve of Ventura Street. Both spans are 49 feet, with a total bridge length of 98 feet.

The UPRR/Ventura Street underpass is situated adjacent to the HST structure and facilitates the crossing of the UPRR line over Ventura Street. The bridge configuration, span lengths, and substructure arrangement are similar to the HST structure. The superstructure, however, is specified as a series of steel girders, with a top steel plate to form the deck.

5.10 G Street / Ventura Street Underpass

The G Street/Ventura Street underpass is a two-span road bridge, each 45 feet, with a total length of 90 feet. The superstructure is composed of a precast, prestressed RC slab, which is supported by two RC abutments and a three-column RC bent situated in the central reserve of Ventura Street.

5.11 Tuolumne Street Crossing

The Tuolumne Street crossing is a five-span road bridge that spans over the HST alignment, UPRR line, and H Street. The bridge has a total length of 760 feet 4 inches, with spans ranging between 100 feet and 160 feet 4 inches. The superstructure is formed of precast, prestressed RC girders and RC deck slab. The substructure is composed of RC abutments and five intermediate RC columns.

5.12 E Church Avenue Crossing

The E Church Avenue crossing is a seven-span road bridge with a total length of 960 feet. It spans over the HST alignment, UPRR and BNSF lines, and Sunderland Ave. The bridge is skewed, with a maximum skew of 30°. The superstructure is formed by precast, prestressed RC girders and RC decks slab. The substructure is composed of intermediate RC bents, and RC abutments, which also form the interface with the MSE approach embankments. To maintain adequate clearance to Sunderland Avenue, the bent cap has been removed from the easternmost bent and instead an RC diaphragm has been specified to transfer the loads from the superstructure to the columns.

5.13 E Central Avenue Crossing

The E Central Avenue crossing is a six-span road bridge spanning over the HST alignment and the adjacent BNSF line. The superstructure is formed with precast, prestressed RC girders and RC deck slab and is supported on intermediate RC bents. The abutments are situated at the interface between the crossing and the MSE approach embankments. The span lengths range between 80 and 110 feet, and the total length of the bridge is 610 feet.

5.14 E American Avenue Crossing

The E American Avenue crossing is a four-span road bridge, with spans ranging between 60 and 130 feet. The crossing bridges over the HST alignment and BNSF line, and has a total length of 380 feet. The superstructure is composed of precast, prestressed RC girders and an RC deck slab. The substructure is made up of intermediate RC bents and abutments formed by RC bearing seats situated on sloped approach embankments.

5.15 Stanislaus Street Crossing

The Stanislaus Street crossing is a six-span road bridge with a total length of 795 feet. The bridge spans over the HST alignment, UPRR line, and H Street. The span lengths range between 120 and 159 feet, and are supported on intermediate RC bents along with RC abutments at the interface between the bridge spans and the MSE approach embankments. The superstructure is formed by precast, prestressed RC girders and RC deck slab.

5.16 Stanislaus Street Pedestrian Crossing

The Stanislaus Street Pedestrian footbridge spans over the HST alignment and UPRR alignment and is situated adjacent to the Stanislaus Street Roadbridge. The superstructure is formed by a precast, prestressed RC box girder and is supported by RC columns. The approach ramp from the east and west abutments are elliptical, in order to attain the required elevations within the construction envelope, while also maintaining an acceptable approach gradient. The span lengths range between 85 and 255 feet.